CHAPTER 4
FOUNDATION DESIGN

4-1. Selection of foundation type. As illustrated figure 4-1, site data, engineering policies, general and environmental criteria, cost limitations, knowledge of the state-of-the-art, and specific facility requirements are used to develop the engineering design. Cost comparisons should be made for realistically competitive alternate designs. Feedback may occur at all stages of the procedure, resulting in a new or modified approaches, design refinements and revised cost estimates, within constraints established by the basic data. Selection is finally made of the foundation type which most effectively meets requirements at minimum cost, and design drawings and specifications are completed for this type. Accurate cost estimates require full development of the design details covered in succeeding chapters of the manual, as applicable. However, the designer should begin to make at least rough cost estimates early in the design process in order to insure that efforts are applied along avenues most likely to produce economical results. In subarctic areas without permafrost, procedures for selection of foundation type are similar to those in seasonal frost areas of the temperate zones except that difficulties and expense involved in preventing uplift or thrust damage from frost heave, as by placing footing below the annual frost zone, are intensified. In permafrost areas, however, the selection of foundation type is more complex; it is rarely practical here to carry footings below the zone of frozen ground and additional factors must be considered in design[86,201]. The principal foundation design options for foundations on permafrost are illustrated in figure 4-2. For areas of deep frost penetration without permafrost the design alternatives are similar, except that the possibility of permafrost degradation does not have to be considered.

a. Construction when foundation supporting conditions will not be adversely affected by thaw.

(1) Whenever possible, structures in arctic and subarctic areas should be located on clean, granular, non-frost-susceptible materials or rock which are free of ground ice. In absence of subsurface exploration, per-

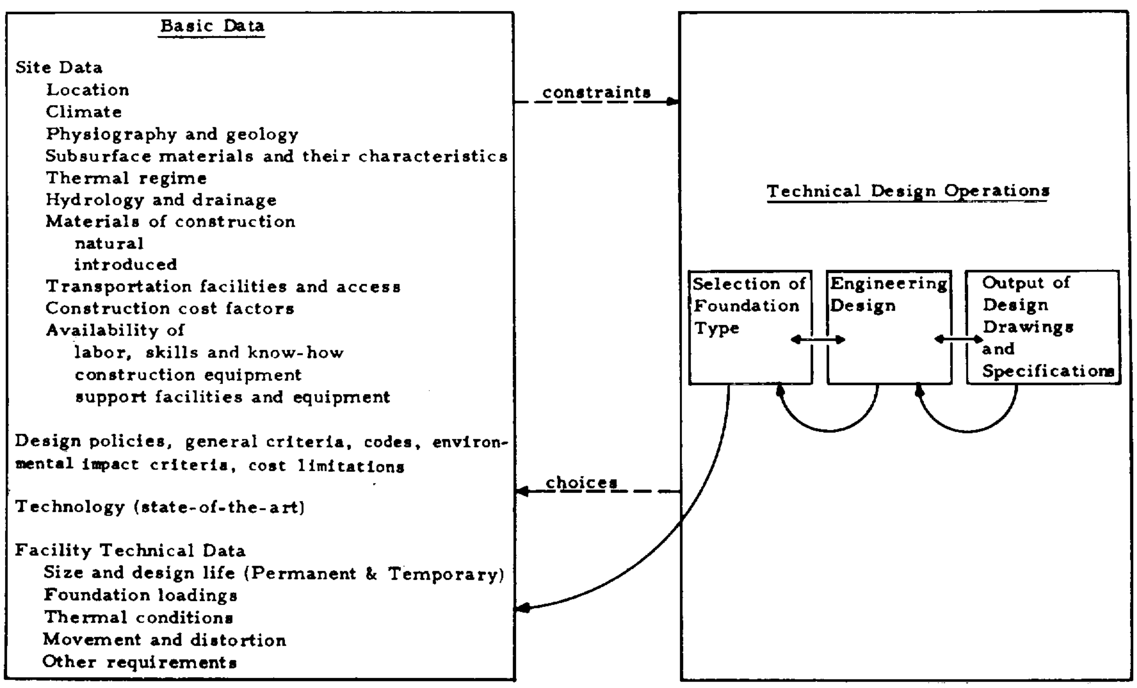

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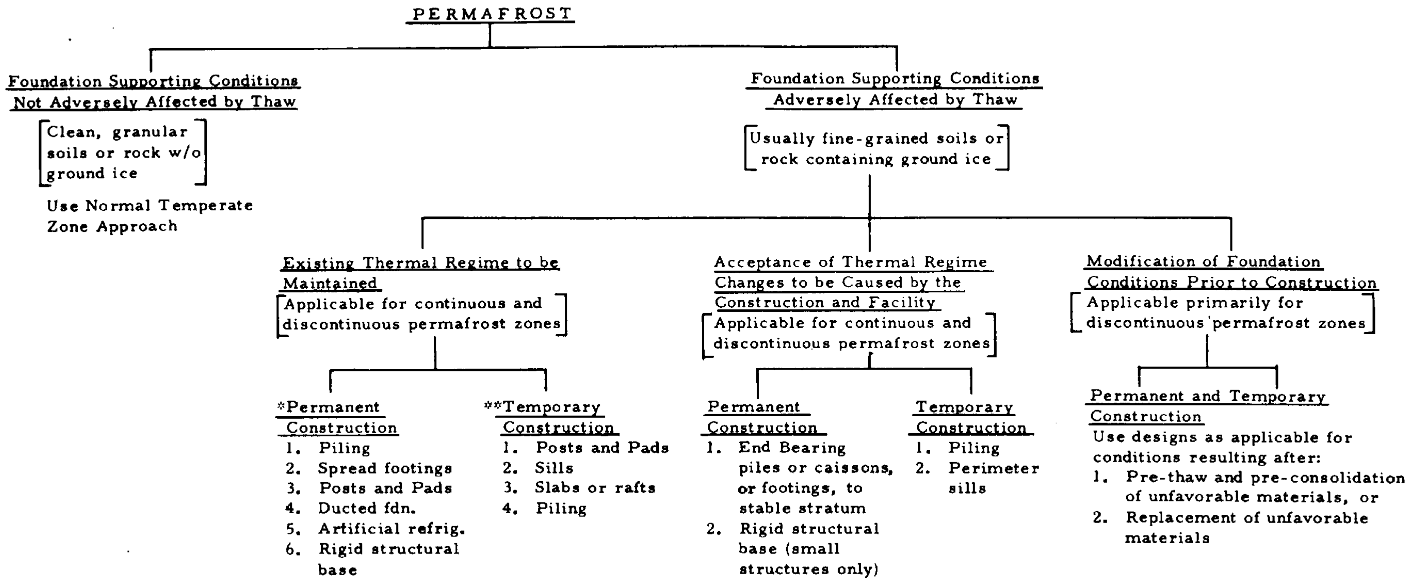
Figure 4-1. Design of foundations in areas of deep frost penetration and permafrost.



As defined in Construction Criteria Manual, DOD4270.1-M, March 1968:

*Permanent Construction - Construction incorporating the type and quality of materials and equipment, and details and methods of construction, which results in a building or facility suitable to serve a specific purpose over a minimum life expectancy of 25 years with normal maintenance.

**Temporary Construction - Construction incorporating the type and quality of materials and equipment, and details and methods of construction, which results in a building or facility suitable to provide minimum accommodations at low first cost to serve a specific purpose for a short period of time, 5 years or less, in which the degree of maintenance is not a primary design consideration.

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Figure 4-2. Design alternatives.

manently frozen sands, gravels, and bedrock cannot be automatically assumed to be free of ice inclusions such as lenses or wedges (para 2-5). However, such foundation materials, free of excess ice, do occur frequently, as in important areas of interior Alaska. When clean sands and gravels, or bedrock free of ground ice, are present, foundation design can frequently be identical with temperate zone practice, even though the foundation materials are frozen below the foundation level. Seasonal frost heave and settlement are comparatively small or negligible in clean, granular, non-frost-susceptible materials under nominal confinement. When such materials thaw they remain relatively stable and retain good bearing characteristics. The tendency of freedraining sand and gravel deposits to have low ground water levels, within limits set by surrounding terrain, contributes to their general desirability as construction sites. It is possible that local sand and gravel deposits may be found quite loose or containing ground ice due to various causes, such as silt inclusions within the soil mass, and some settlements may occur at such points if thawed materials are reconsolidated under the effects of loading and/or vibrations. Whether or not such conditions are present in significant degree must be determined in the course of the site investigations, and whether or not they need be taken into account in the design and construction will depend in part on the type and importance of the structure. Often, measures to preserve permafrost are unnecessary in construction on deep, clean sand and gravel deposits. Figure 4-3 shows, for example, the very minor settlement which accompanied thaw progression under a three story reinforced concrete

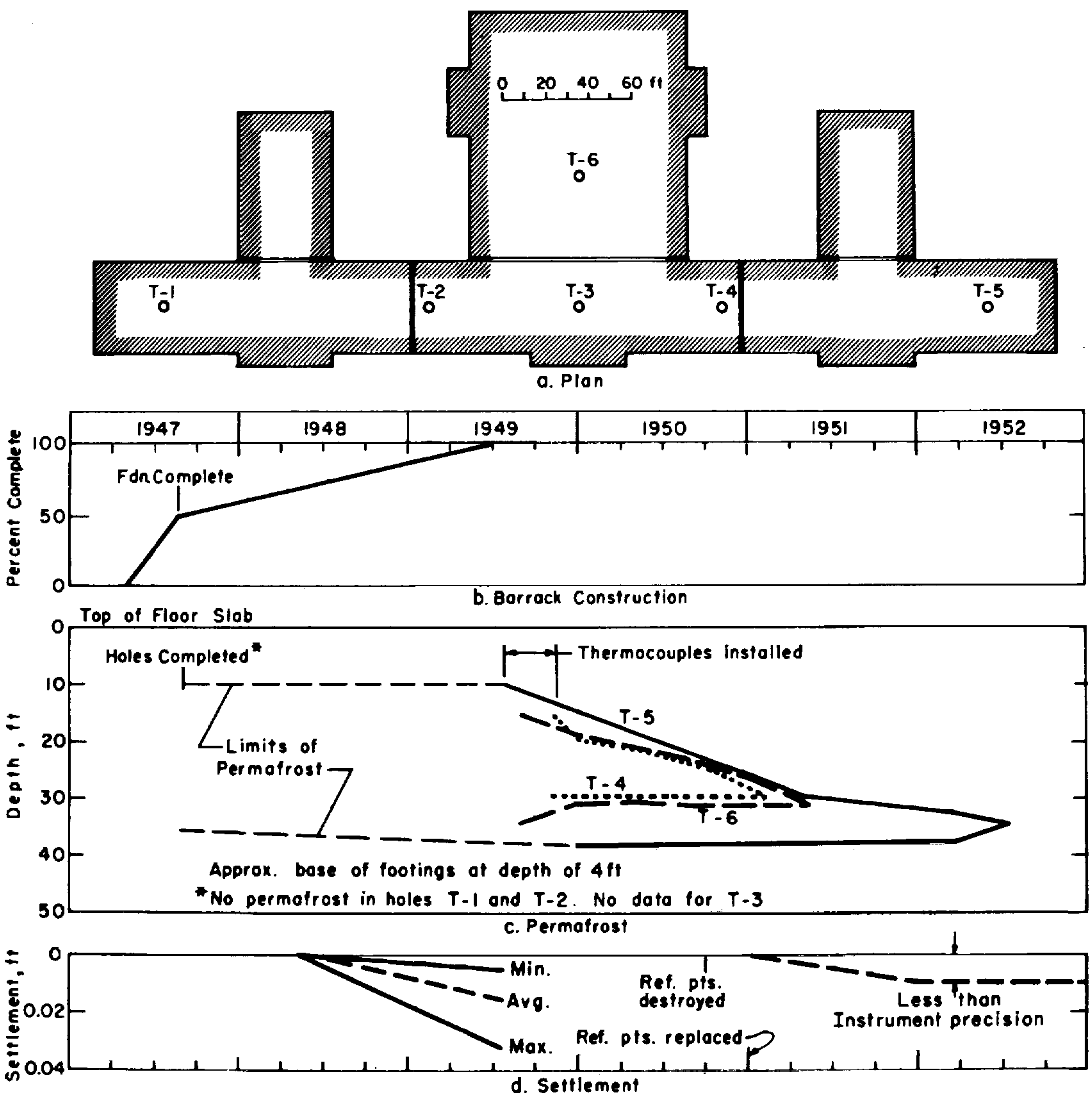

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Figure 4-3. Thawing of permafrost under 3 story, reinforced concrete, 500-man barracks, Fairbanks,. Alaska. Reference points consisted of bolts installed in outer side of foundation wall above ground[100].

building at Ft Wainwright (Ladd AFB), Fairbanks, Alaska.[100] No adverse effects could be detected. The settlement indicated by the earlier set of reference points in figure 4-3d may be attributed to compression of 2 to 6 feet of gravel backfill which had been placed beneath the footings and of the 3 to 4 feet of underlying gravelly soil which was at that time thawed to a depth of 10 feet. In special cases, such as of very important or critical structures which can tolerate only minute settlement or which transmit significant vibratory stresses to the foundation, or where effects of thaw after construction would be otherwise unacceptable, it may be necessary to employ pre-thawing (b below) followed by foundation soil consolidation and/or stabilization in accordance with the same principles and techniques as applicable under similar situations in non-frost areas.

(2) In some cases, in areas of low precipitation, fine-grained soils may be encountered which are free of ground ice and sufficiently dry and compact so that they may in theory be treated in the same way as granular non-frost-susceptible soils. However, the possibility that moisture may be introduced into such soils later, during or following construction such as from roof drains, dry wells or condensate discharge, must be considered.

b. Construction when foundation supporting conditions will be adversely affected by thaw. Permafrost in which the soil is fine-textured or contains significant fractions of silt or clay frequently contains significant amounts of ground ice in various forms such as lenses, veins, or wedges. Bedrock also often contains substantial ground ice. Any change from natural conditions which results in a warming of the ground can result in progressive lowering of the permafrost table over a period of years, known as degradation. Thawing of high ice content materials may produce large volume reduction and settlement of overlying soil and structures. Consolidating soils may have greatly reduced shear strength. Degradation subsidence in soils containing ground ice is almost invariably differential and hence potentially very damaging to a structure. The local thaw-depression produced in the permafrost will tend to form a collection sump for ground water, and underground components of the construction may encounter a difficult water control problem. Under some conditions lateral soil movements may develop. Degradation may occur not only from building heat but also from solar heating, in positions which sunlight can reach, from ground water flow, and from heat from underground utility lines. During the winter, seasonal freezing of frost-susceptible materials may produce substantial frost heaving. For locations in areas of fine textured soils, design should consider the following alternatives, as shown in figure 4-2.

Maintenance of existing thermal regime.
Acceptance of the changes in the thermal regime and foundation conditions which will be caused by the construction and facility and allowance for these in design. Modification of foundation conditions prior to construction. This includes the alternatives of removing and replacing unacceptable foundation conditions, and thawing to eliminate permafrost.

The principles of these three alternative methods are explained in the following paragraphs.

(1) Existing thermal regime to be maintained.

(a) This design approach is applicable for both continuous and discontinuous permafrost zones.

(b) In surface construction, it is possible to utilize the low temperatures of the freezing season to maintain permanent frozen soil conditions in the finegrained soil at and below the depth of the foundation support by providing for circulation of cold winter air through a foundation ventilation system or by some other method of foundation cooling. In some circumstances artificial refrigeration systems may be necessary.

(c) In order for natural cooling methods to be practical, it is necessary to cool the upper foundation soils sufficiently during the winter so that the foundation materials thawed in the preceding summer will be completely refrozen, progressive annual lowering of the permafrost table will be prevented, and there will be sufficient "storage of cold" so that maximum temperatures of permafrost do not exceed limits for safe foundation support. The latent heat of fusion of the ice produced by the winter refreezing of the moisture contained in the upper soil layers will be a major factor in restricting summer thaw to a shallow depth. When seasonal frost heave and settlement of the soil under the structure must be controlled and summer thaw must be prevented from reaching into underlying unsatisfactory foundation materials, sufficient thickness of non-frostsusceptible granular material should be placed to achieve the desired effect. The flow of heat from a building to the permafrost is retarded, and the refreezing of foundation materials aided, by placing insulation between the floor of the building and the underlying foundation ventilation or cooling system.

(d) To minimize disturbance of the subsurface thermal regime, the existing vegetative cover and seasonal frost zone material. should be protected and preserved in non-work are. In the areas of actual construction, however, a mat of granular non-frostsusceptible material should be placed over soft vegetative cover to serve as a working surface, unless the work can be accomplished in winter without essential damage to the surface materials. Since it is generally not feasible to remove such a mat later and restore the vegetative cover to its original condition, the mat should

be designed as a permanent feature of the facilities. Mat thickness criteria are discussed in paragraph 4-2. Many types of fibrous organic surface layers when of sufficient thickness will support a few coverages of light construction equipment, but low-strength surface materials may require end-dumping techniques even to enable placement of the working mat. However, such mat or fill is not by itself a complete design solution when placed over frozen, highly compressible or high ice content deposits if there is any possibility of subsequent permafrost degradation. In order to estimate the structural properties of the permafrost in its frozen state, the temperatures at which it will be maintained must be estimated.

(e) It will be apparent that maintenance of the existing thermal regime is much easier to achieve in areas of continuous permafrost where permafrost temperatures are low than in the discontinuous and borderline permafrost areas where there is less margin of safety and greater care is required in design analysis.

(2) Acceptance of thermal regime changes to be caused by the construction and facility.

(a) This design approach is applicable for both continuous and discontinuous permafrost zones.

(b) If small progressive thawing is anticipated in the permafrost, settlement of structures may be avoided by supporting them on piles which are frozen into permafrost to a depth that is well below the level of anticipated degradation during the planned life of the structures and that is also sufficiently deep to resist any heaving forces during winter periods; it is approach is usually only used for temporary structures such as construction camp buildings and the possibility of unacceptable environmental impact must be considered. Piles or caissons may also be designed for end bearing on icefree bedrock or other firm, stable underlying formation. This method is particularly feasible when the finegrained foundation soils containing ground ice form a relatively shallow cap. Designing for end-bearing is a very good approach for bridge piers or similar structures where foundation ventilation or similar systems are not practical. It must be kept in mind that once a residual thaw zone has developed as a result of the construction, the temperature of the underlying permafrost, and its structural capacities for members such as piles, will be seriously altered.

(3) *Modification of foundation conditions prior to construction.*

(a) This design approach is applicable almost solely in the discontinuous or borderline permafrost areas. It has only very limited applicability for areas of continuous, low-temperature permafrost.

(b) Under this alternative, one procedure would be to compute the expected final extent of thawed or unfrozen foundation materials produced by construction and subsequent facility operation and to pre-thaw and pre-consolidate the foundation within this zone. Thawing techniques are discussed in paragraph 6-2. One major disadvantage of this scheme lies in the difficulty of accurately anticipating the new thermal regime or thaw bulb position that will be stable, particularly if permafrost is too thick to be thawed completely through; continuing thaw of permafrost could result in settlement, but refreezing at the boundaries of pre-thawing would tend to produce heave. If only a relatively shallow layer of frozen fine-grained soil exists in or on an otherwise satisfactory granular foundation, the scheme may be more practical. The Corps of Engineers has constructed successfully performing facilities at both Anchorage and Fairbanks, Alaska, in which the major portions of frost-susceptible soils have been prethawed, consolidated and utilized in place with adequate heat permitted to escape to insure continuous thawed conditions. However, even under relatively favorable conditions, refreezing of the foundation when the building is vacated and heating discontinued for an extended period can cause major facility damage under this scheme. Because possible changes in building usage over long periods are relatively unpredictable and communication of requirements for continuous facility heating to successor occupants cannot be relied upon, this approach should not be used except with specific approval of HQDA (DAEN-ECE-G), WASH DC 20314.

(c) The same risk also occurs if a foundation cooling system is installed to stabilize the thawed regime of a foundation where degradation has already been experienced. At a regional school at Glenallen, Alaska, frost heave and structural difficulties, including differential movement of 2 inches, was apparently caused by operation of a mechanical refrigeration system for cooling under-floor air at temperatures low enough to cause progressive refreezing of underlying thawed soil.[111]

(d) Where the fine-grained settlement susceptible permafrost soils are limited to a relatively shallow upper layer, say up to about 20 ft thick, and clean, granular, non-settlement-susceptible soils underlie, it may be feasible to remove the undesirable soils and replace them with compacted fill of clean, granular soils. Design and construction may then follow normal temperate zone techniques. The U.S. Army Engineer District, Alaska, has used this technique successfully at Fairbanks, Alaska.

(e) Occasionally it may be possible to alter surface conditions at a construction site up to several years in advance so that adjustment of the thermal balance may occur naturally over a long time.

(f) Where permafrost is to be pre-thawed, the relative density of the soil in place, after thaw, should be

estimated together with related effects of the changes on subdrainage in the area and the thermal regime in the ground.

(g) Preferred practice is to aim as closely as practicable at the method in (1) above, but with knowledge that construction must inevitably effect some changes in accordance with the method in (2) above. Under the latter, design should aim at making the changes in thermal regime determinate. Where conditions are favorable, the method in (3) above may sometimes obviate the need for special foundation structural design, although the requisite conditions for employing this technique occur somewhat rarely.

(h) Unless foundation soils are clean granular materials which will not produce significant frost heave or settlement with fluctuations of thermal regime, it is accepted practice to support structures either entirely on top of the annual frost zone or entirely in the underlying permafrost zone using piles or other means to transmit structure loads through the annual frost zone.

c. Simplified example of selection of foundation type in an area of discontinuous permafrost.

(1) *Facility Requirements*

One story permanent facility, above surface. 250 Ib/ft' minimum floor load capacity.

72° F normal room temperature.

No special thermal loads.

(2) *Site Data*

Within 5 miles of a city, in discontinuous permafrost region.

Construction materials, labor, equipment, transportation all readily available.

Clean, bank run gravel borrow available 3 miles from site.

Mean permafrost temperature 300F.

Thawing index = 5700.

Permafrost thickness = 200 ft continuous over site.

Overburden: 5 ft silt, non-plastic ML-Vs, over clean, frozen, thaw-stable sandy gravels, GW-Nbn, extending to bedrock at 210 feet.

No ice wedges. Anticipated settlement of silt on thaw = 1 1/2 in./ft.

(3) Since there is only 5 feet of thaw-susceptible over-burden, floor loading is high, and gravel is available, the silt should be removed and replaced with gravel, and a slab-on-grade type foundation should be employed. For a facility in which a more modest floor load capacity would be acceptable, a basement-type construction might be considered since this would avoid the hauling, spreading and compaction of gravel. However, a basement water problem might be encountered if thaw water were unable to drain naturally from the thaw bulb which will develop under the structure.

d. Simplified example of selection of foundation type in an area of continuous permafrost.

(1) Facility Requirements

One story permanent facility, above surface.

40 lb/ft' floor load capacity.

72°F room temperature, year-round No special thermal loads.

(2) Site Data Very remote site, continuous permafrost.

No local trained labor or materials.

Mean permafrost temperature + 12° F.

Thawing index = 700.

Freezing index = 8000.

Permafrost thickness = 1700 feet.

Overburden: 90 ft glacial till, silty gravel GMVr, containing ice wedges, over bedrock.

Anticipated settlement of overburden on thaw = 2 in./ft of thaw depth.

(3) Because permafrost is very deep and continuous, as well as containing substantial ground ice, the alternative "Modification of Foundation Conditions Prior to Construction" (fig. 4-2) is impractical and inapplicable. Permafrost temperature is low enough so that a thermally stable design is readily achievable. Under the foundation conditions, the alternative "Acceptance of Thermal Regime Changes to be Caused by Construction and Facility" is impractical for a permanent facility. Therefore, the possible designs shown under "Existing Thermal Regime to be Maintained" should be considered. For the light floor loading the ducted foundation and the rigid structural base options are too heavy and costly and are inappropriate. Since there is no special thermal load, permafrost temperature is low, and the structure is above-surface and can have a ventilated foundation, there is no need for artificial refrigeration. Therefore, design alternatives for permanent type foundation are piling, spread footings, and post and pad. Choice can be made on basis of cost after development of details for each of these types to the degree needed for resolution.

4-2. Control of heat transfer and degradation.

a. General.

(1) Frost and permafrost conditions, thermal regime in the ground and effects of heat from facilities have been discussed in general terms in paragraphs 1-2 and 2-1. Beneath and surrounding a foundation on frozen soil, the degree of disturbance of the normal thermal regime brought about by construction depends upon such factors as construction methods, exposure, drainage, snow cover and drifting, and extent of disturbance or change of the original surface cover, in addition to normal heat loss from the structure which may reach the ground. These factors must be taken into account in estimating both the immediate and long term stability of the struc-

ture foundation. Changes in the thermal regime in turn produce corresponding changes in such factors as the strength and creep properties of the foundation media and subsurface drainage. These factors are of far greater importance to foundation stability in the marginal areas of relatively warm, discontinuous permafrost than in the areas of either cold, continuous permafrost or of deep seasonal frost.

(2) In both seasonal frost and permafrost areas, heat flow should also be considered in relation to discomfort. from cool floors, the cost in added fuel requirements of undue heat loss, and the possible desirability of some heat loss to assist in protecting against frost heave of footings.

(3) Large heat-producing structures, particularly steam and power plants, present an especially serious foundation design problem because of the potential large and continuous flow of heat to the foundation. Heavy floor loadings often associated with such facilities may make it expensive to provide ventilation beneath the floor. The problem is commonly further complicated when severe dynamic loadings occur, such as from generator equipment. In addition, proper operation of such a plant may be seriously impaired by any differential floor movements. For these reasons, such structures should, whenever possible, be located on non-frost-susceptible granular soils in which effects of thawing or frost action will not be detrimental (making sure that the granular soil is not simply a relatively shallow layer covering fine-grained soils containing ground ice). At heat producing facilities it is essential to make specific provisions for disposal of warm water waste so that degradation of permafrost will not be caused by discharge of such water under or adjacent to the foundation. Care must be taken to avoid leakage from water or steam distribution lines and of deflection against the ground of warm air from facility ventilating systems. Whenever possible, heat producing plants should be housed in independently located buildings if they might be sources of differential thawing and subsidence for connected or closely adjacent facilities.

(4) Thermal stability and potential frost action in foundations of unheated facilities such as bridge piers, storage igloos, tower footings, loading platforms, and exterior shelter areas must also be analyzed carefully. In seasonal frost areas absence of an artificial heat source in an unheated facility, combined in some facilities with the shading effect of the upper parts of the structure, will usually result in maximum potential frost penetration, maximum frost adhesion to the foundation, and maximum tendency toward frost heave. In permafrost areas, on the other hand, thermal stability of the permafrost is much easier to achieve in foundations of unheated than heated facilities.

(5) The designer must keep in mind that disturbance of the natural ground surface by construction efforts will normally cause some change in the position of the permafrost table, even though a continuously degrading condition may not be produced. In borderline permafrost areas it may be necessary to use vegetation, reflective paint, or shading devices to assist in obtaining a stable permafrost condition for the new construction.

(6) Serious difficulties may also occur if facilities in permafrost areas designed for no heat or a relatively low heating level are converted to higher heating temperatures; the results may be degradation of permafrost and foundation settlement. During design the possibility of future higher heating temperatures in facilities must be examined; if there is substantial probability of such future conversion, design for the higher temperature levels should be seriously considered.

(7) POL and water tanks should have ventilated foundations when located on permafrost subject to settlement on thaw. Water storage tanks are always kept above freezing and if placed directly on the ground, would cause continuous heat input into a frozen foundation even though insulated. POL may be loaded into storage tanks at relatively elevated temperatures, giving off considerable heat while cooling; also heavy oils may have to be heated for pumping.

(8) In pile foundations the piles themselves are also potential conductors of heat from the building or from warm air or sunlight to which they are exposed in the summer into the foundation but this is seldom a real problem because most conducted heat is diffused from the pile into the air ventilation space in winter or into the annual freeze and thaw zone, within a distance of 2 or 3 diameters along the pile. Probing and test pitting have shown slightly deeper summer thaw directly adjacent to unpainted steel piles which are exposed above ground to heating by both direct sunlight and air, but the amount has not been found to exceed about 12 to 18 inches in depth for piles properly installed and is generally much less. However, even this effect can be minimized with skirting, white paint or radiation shields where needed as discussed in f below.

(9) Care should be taken in designing foundations for refrigerated warehouses, refrigerated fuel tanks or similar foundations to avoid frost heave from progressive freezing of underlying soils. Such effects may take years to become evident. It should be noted that insulation only slows such effects; it does not prevent them.

(10) Detailed procedures for foundation thermal computations are presented in TM 5-852-6/AFM 88-19, Chapter 6[14].

b. Estimation of ordinary freeze and thaw penetration.

(1) Design depth of frost penetration.

(a) In areas of seasonal frost conditions, the design depth of seasonal frost penetration for situations not affected by heat from a structure should preferably be the maximum found by actual measurement under conditions representative of those for the facility design, or by computations if measurements are not available. When measurements are available, they will frequently need to be adjusted by computations to the equivalent of the freezing index selected as the basis for design, as measurements may not be available for a winter having a severity equivalent to that value. The air freezing index to be used in the estimate of frost penetration should be selected on the basis of the expected life of the structure and its type. For average permanent structures, the air freezing index for the coldest year in 30 should be used; this is more conservative than the coldest-year-in-ten (or average of 3 coldest in 30) criterion used for pavement design because permanent buildings and other structures are less tolerant of vertical movement than pavements. For structures of a temporary nature or otherwise tolerant of some foundation movement, the air freezing index for the coldest year in ten or even the mean air freezing index may be used, as may be most applicable.

(b) For average conditions, the air freezing or thawing index can be converted to surface index by multiplying it by the appropriate n factor from table 4-1.

(c) The frost penetration can then be computed using the detailed guidance given in TM 5-852-6/AFM 88-19, Chapter 6 [14]. Approximate values of frost penetration may also be estimated from figure 4-4b for

Table 4-1. n - Factors for Freeze and Thaw (Ratio of Surface Index to Air Index)[14]

Type of Surface (a)	For Freezing Conditions	For Thawing Conditions
Snow Surface	1.0	-
Portland Cement Concrete	0.75	1.5
Bituminous Pavement	0.7	1.6 to 2+ (b)
Bare Soil	0.7	1.4 to 2+ (b)
Shaded Surface	0.9	1.0
Turf	0.5	0.8
Tree- covered	0.3(c)	0.4

(a) Surface exposed directly to sun and/or air without any overlying dust, soil, snow or ice, except as noted otherwise, and with no building heat involved.

(b) Use lowest value except in extremely high latitudes or at high elevations where a major proportion of summer heating is from solar radiation.

(c) Data from Fairbanks, Alaska, for single season with normal snow cover permitted to accumulate.

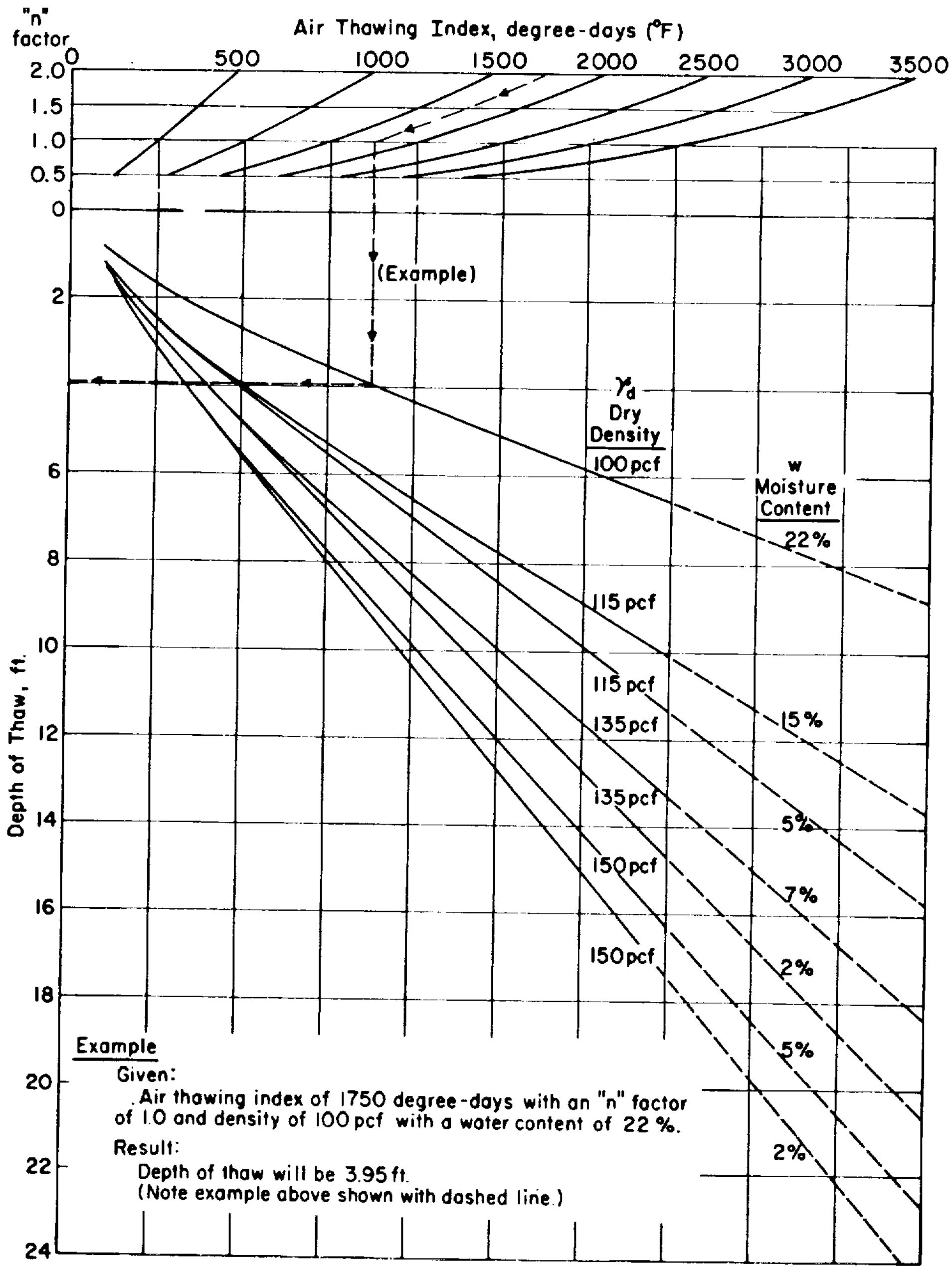

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Figure 4-4a. Approximate depth of thaw or freeze vs air thawing or freezing index and n-factor for various homogeneous soils. In calculations for the curves, thermal conductivities for frozen and thawed states have been averaged together. Because the actual effective thermal conductivity may not be equal to this average value during either freezing or thawing, precise agreement between measured and predicted values should not be anticipated. However, deviations due to this approximation should not exceed those arising from other causes. Curves developed from calculations based on procedures in TM 5-852-6[14]. (Air thawing index vs depth of thaw.)

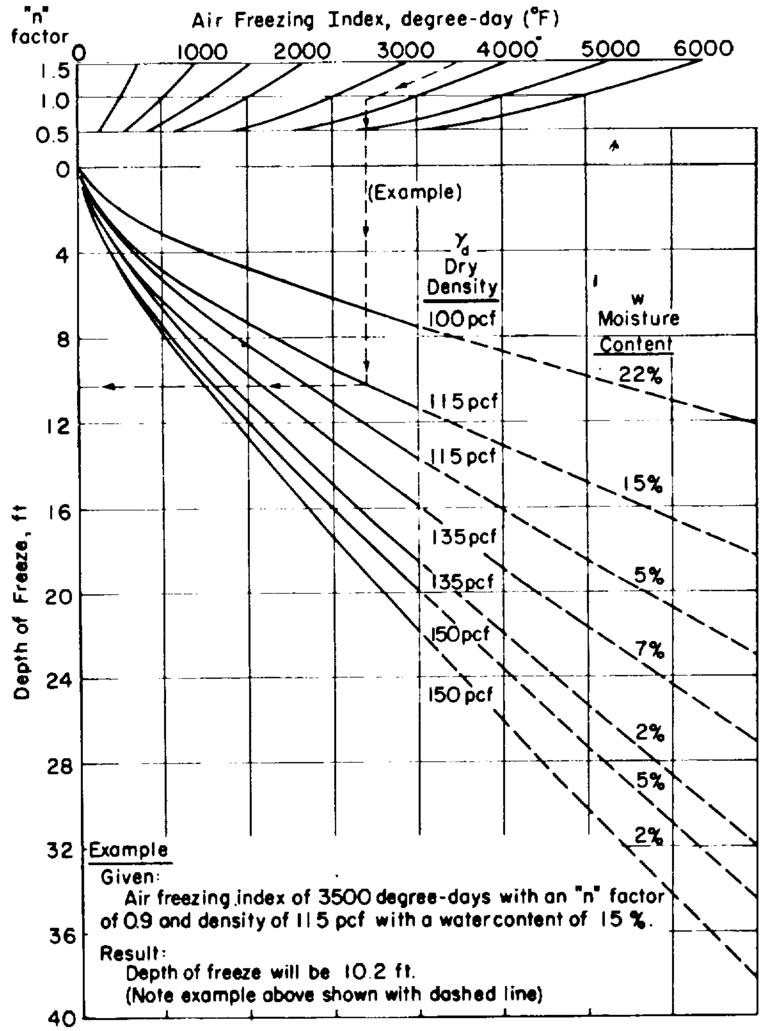

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Figure 4-4b. Approximate depth of thaw or freeze vs air thawing or freezing index and n-factor for various homogeneous soils. In calculations for the curves, thermal conductivities for frozen and thawed states have been averaged together. Because the actual effective thermal conductivity may not be equal to this average value during either freezing or thawing, precise agreement between measured and predicted values should not be anticipated. However, deviations due to this approximation should not exceed those arising from other causes. Curves developed from calculations based on procedures in TM 5-852-6[14]. (Air freezing index vs depth of freeze.)

soils of the density and moisture content ranges there represented. For paved areas kept free of snow, depth of frost penetration may also be estimated from TM 5-818-21 or TM 5-852-3[12], entering the appropriate chart with air freezing index directly.

(d) For given soil conditions, the greatest depth of penetration will be for paved areas not affected by any artificial heat, shaded from the sun, and kept cleared of snow. For heated buildings, both slab on grade and basemented, the heat flowing outward from the foundation tends to modify frost penetration next to the foundation wall. However, a variety of possible situations exists. A building with a basement offers a different condition than one with slab-on-grade construction, and use of insulation or firming on basement or perimeter walls will change heat flow.

(e) Penetration depths for paved areas will nearly always need to be determined by computation rather than from measurements. A deep snow cover may entirely prevent frost penetration; however, the effect of snow cover should usually be disregarded for design purposes, as snowfall may be very small or negligible in the years when temperatures are coldest. Turf, muskeg, and other vegetative covers also help substantially to reduce frost penetration. Some additional guidance on effects of surface conditions is contained in TM 5-852-6/AFM 88-19, Chapter 6[14].

(f) In the more developed parts of the cold regions, the building codes of most cities specify minimum footing depth, based on many years of local experience; these depths are invariably less than the maximum observed frost penetrations. The code values should not be assumed to represent actual frost penetration depths. Such local code values have been selected to give generally suitable results for the types of construction, soil moisture, density, and surface cover conditions, severity of freezing conditions, and building heating conditions which are common in the area. Unfortunately, specific information on how these factors are represented in the code values is seldom available. The code values may be inadequate or inapplicable under conditions which differ from those assumed in formulating the code, especially for unheated facilities, insulated foundations, or especially cold winters. Building codes in the Middle and North Atlantic States and Canada frequently specify minimum footing depths in the range of 3 to 5 feet. If frost penetrations of this order of magnitude occur with fine silt and clay type soils, 30 to 100 percent greater frost penetration may occur in well-drained gravels under the same conditions. With good soil data and a knowledge of local conditions, computed values for ordinary frost penetration, unaffected by building heat, may be expected to be adequately reliable, even though the freezing index may have to be estimated from weather data from nearby stations. In remote areas, reliance on computation of the design frost depth for the specific local conditions at the proposed structure location may be the only practicable or possible procedure, as opposed to reliance on measurements.

(2) Design depth of thaw penetration. Seasonal thaw penetration in permafrost areas typically begins in May or June and reaches maximum depth in the ground in the period July September, as illustrated in figures 4-5 and 4-6. Under paved areas exposed to sunshine, particularly black bituminous pavements,

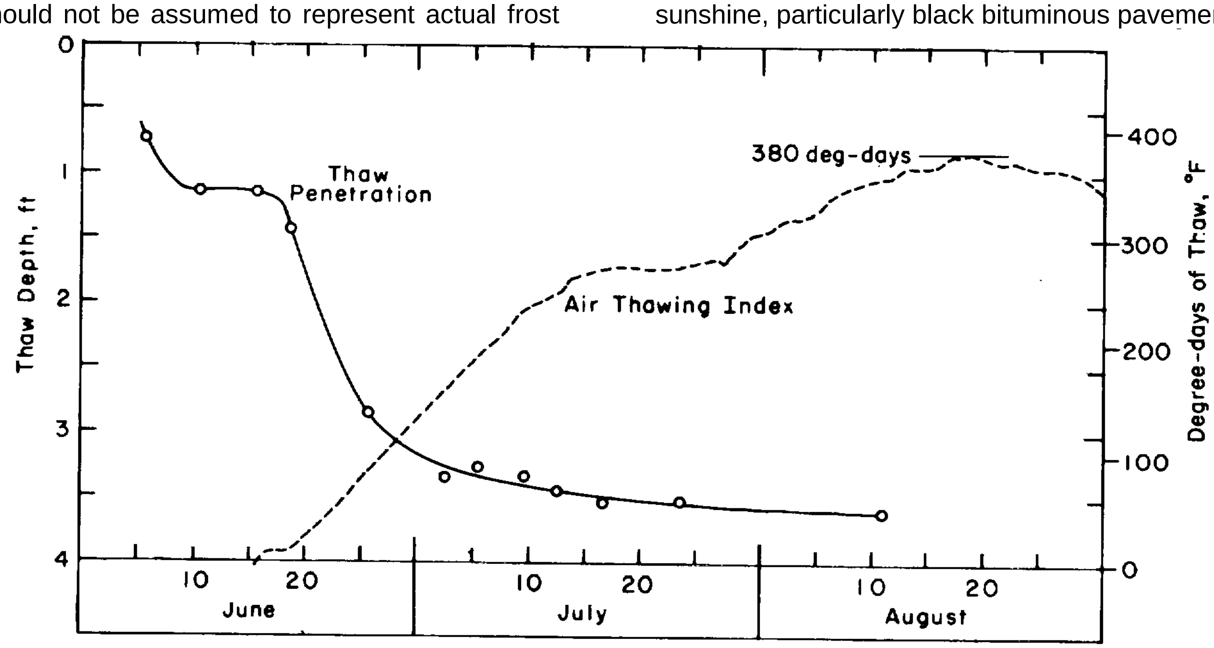

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Figure 4-5. Thaw progression under undisturbed surface, Camp TUTO (near Thule air Base), Greenland. Soils data: average gradation SC, clayey sand; dry unit weight 120124 lb/ft^3; moisture content 8-12 percent, essentially no vegetative cover on surface (by CRREL).

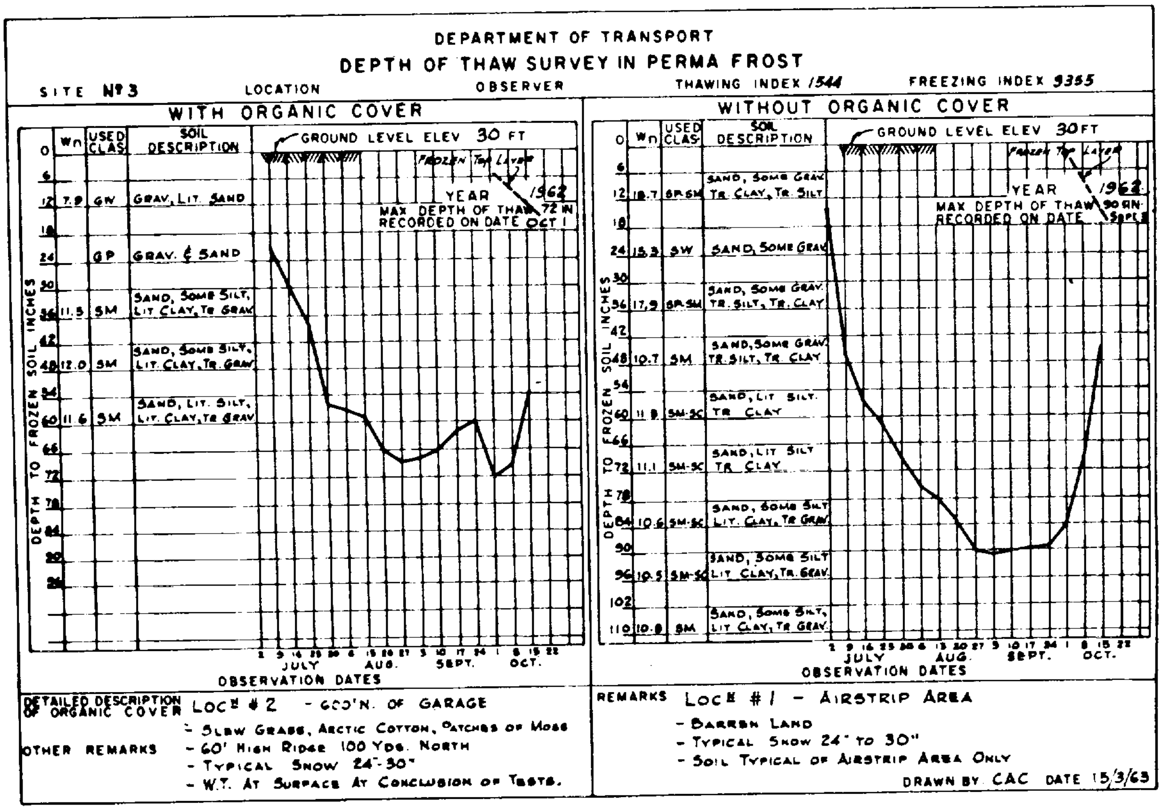
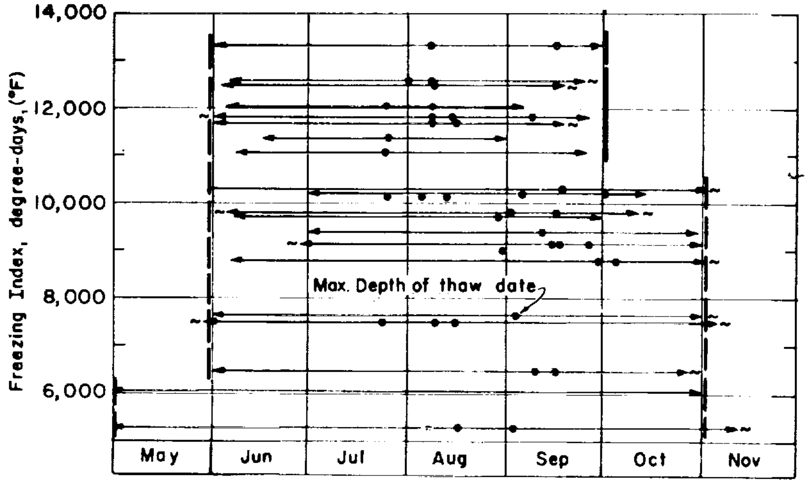


Figure 4-6a. Thaw vs time. Canadian locations (after Sebastyan[188]). (Depth of thaw survey in permafrost, Canadian location)

(Courtesy of Building Research Advisory Board, NAS-NRC)



(Courtesy of Building Research Advisory Board, NAS-NRC)

Figure 4-6b. Thaw vs time, Canadian locations (after Sebastyan[188]). (Period of thaw vs freezing index. Data based on depth of thaw determined by soundings at 38 locations in northern Canada.)

seasonal thaw penetration in high density, extremely well-drained granular materials may be substantial and in marginal permafrost areas may reach as much as 20 feet. Thaw depths under non-paved areas reach typical values as illustrated in figure 4-7 and thaw may vary seasonally from place to place on an airfield site as shown in figure 4-8. The air thawing index to be used in the estimate of seasonal thaw penetration should be established on the same statistical basis as outlined in (1) above for seasonal frost penetration. The air thawing index can be converted to surface thawing index by multiplying it by the appropriate thawing-conditions n-factor from table 4-1. The thaw penetration can then be computed using the detailed guidance given in TM 5-852-6/AFM 88-19, Chapter 6[14]. Approximate values of thaw penetration may also be estimated from figure 4-4a for soils of the density and moisture content ranges there represented. If average annual depth of thaw exceeds average annual freeze depth, degradation of the permafrost will result.

c. Estimation and control of thaw or freeze beneath structures on permafrost.

(1) *General.*

(a) Heat flow from the structure is a major consideration in the design of a foundation in a northern area. Only when no settlement or other adverse effects will result can heat flow from the structure to the underlying ground be ignored as a factor in the long term structural stability.

(b) Figure 4-9 presents an idealized diagram of the effect of size on both total depth of thaw and rate of thaw under a heated structure placed directly on frozen material. Thawing of uniformly distributed ground ice under a uniformly heated structure proceeds most rapidly near the center of the structure and more slowly at the perimeters, tending to produce a bulb-shaped thaw front and dish-shaped settlement surface. Interior footings in such a structure tend to settle progressively in the same dish-shape, at about the same rate as the melting of the ice. However, a rigid foundation slab tends to develop a space under it, at least for a time, after which abrupt collapse may occur. The larger the structure the larger the potential ultimate depth of thaw; however, in initial stages of thaw, the rate of thaw advance under the center of the structure is not a function of structure size. For small temporary buildings it is seldom necessary to completely preclude differential seasonal movements even though it'may be relatively easy to do this; most construction camp buildings, for example, can be maintained easily and the movements brought about by frost action and thaw can be equalized by the use of jacks and shims.

(c) For large structures intended for long term use, maintenance requirements must be kept at a much lower level, consequences of progressive thawing may be more severe, and achievement of adequate ground cooling and thaw depth control with a foundation ventilation system is more difficult.

(2) Building floor placed on ground. When the floor of a heated building is placed directly on the ground over permafrost, the depth of thaw is determined by the same method as that used to solve a multilayer problem when the surface is exposed to atmospheric effects, as explained in TM 5-852-6/AFM 88-19, Chapter 6 [14], except that the thawing index is replaced by the product of the time and the differential between the

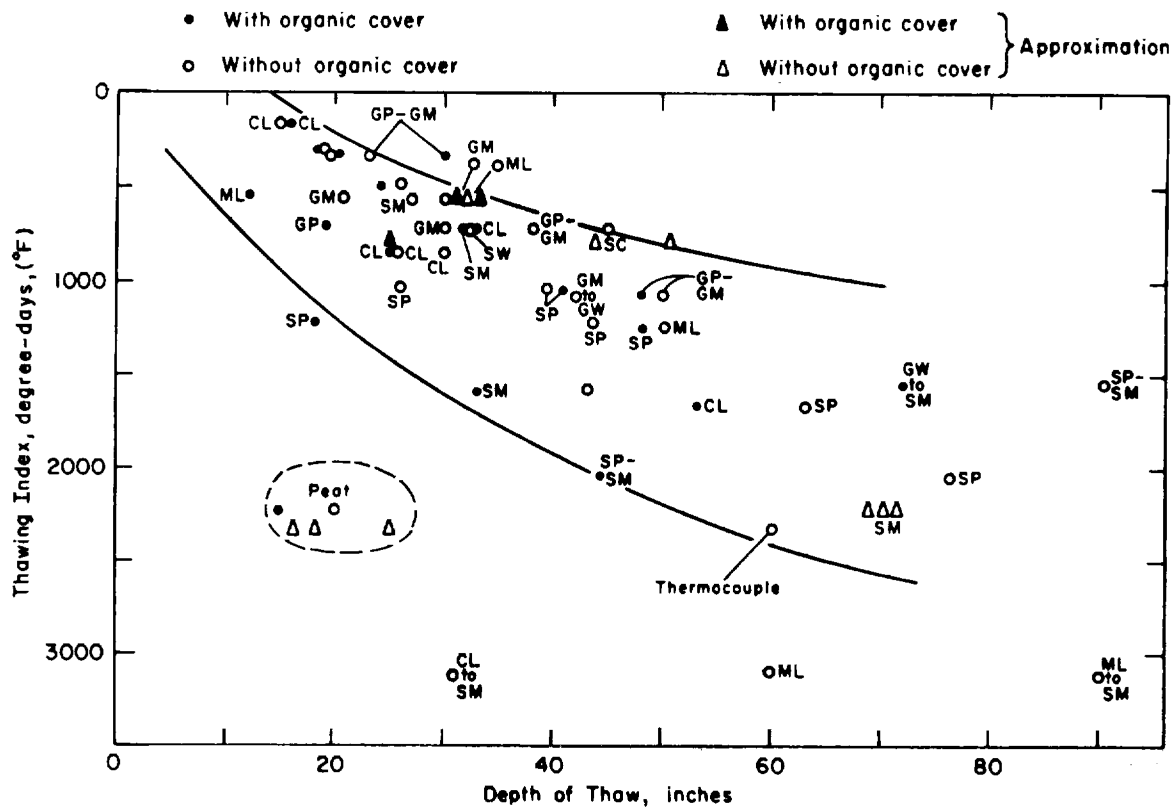


Figure 4-7. Depth of thaw vs air thawing index for unpaved surfaces. Data based on depth of thaw determination by sounding at 62 locations in the Canadian North[188].

(Courtesy of Building Research Advisory Board, NAS-NRC)

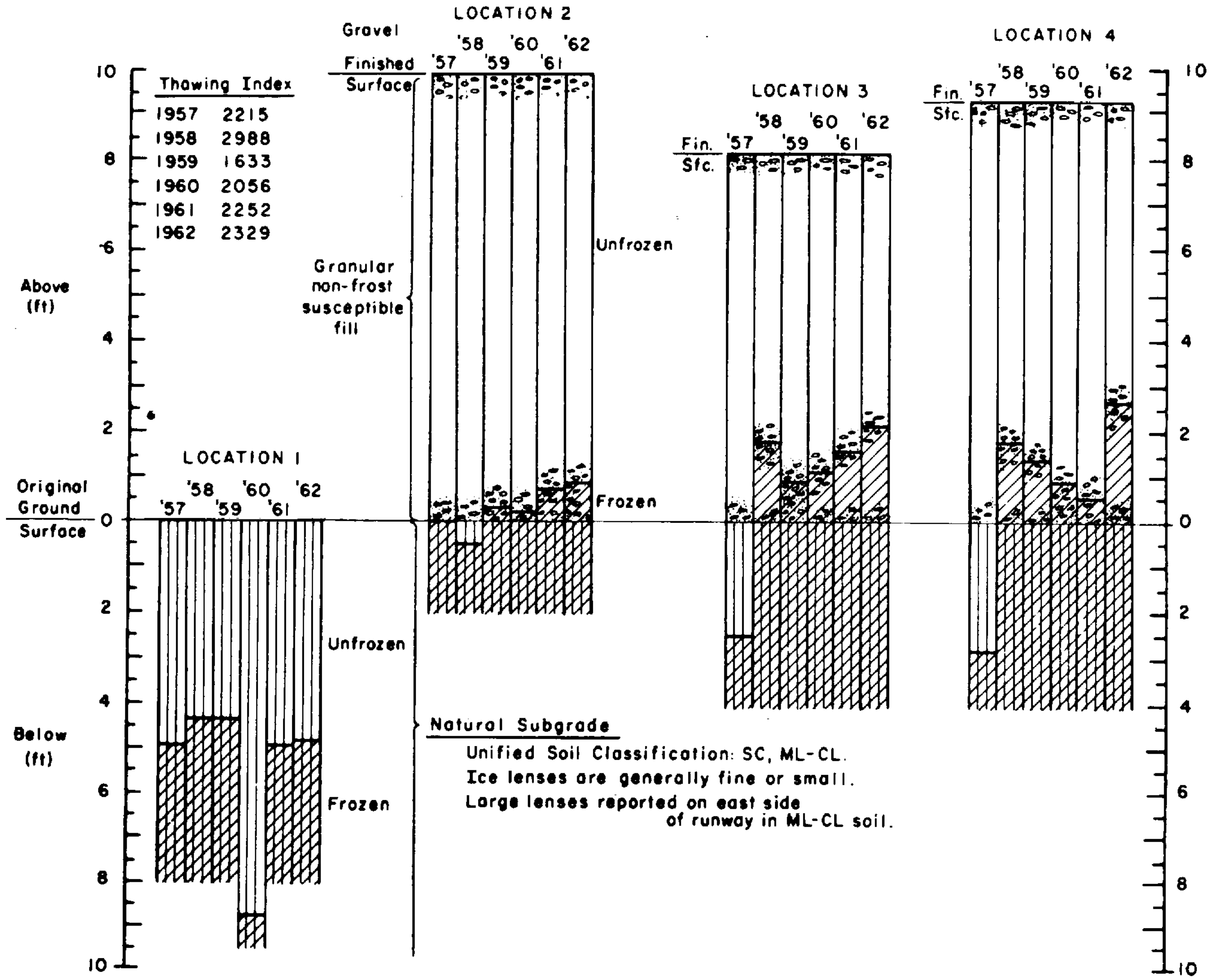


Figure 4-8. Thaw depth as affected by runway construction on permafrost[188].

(Courtesy of Building Research Advisory Board, NAS-NRC)

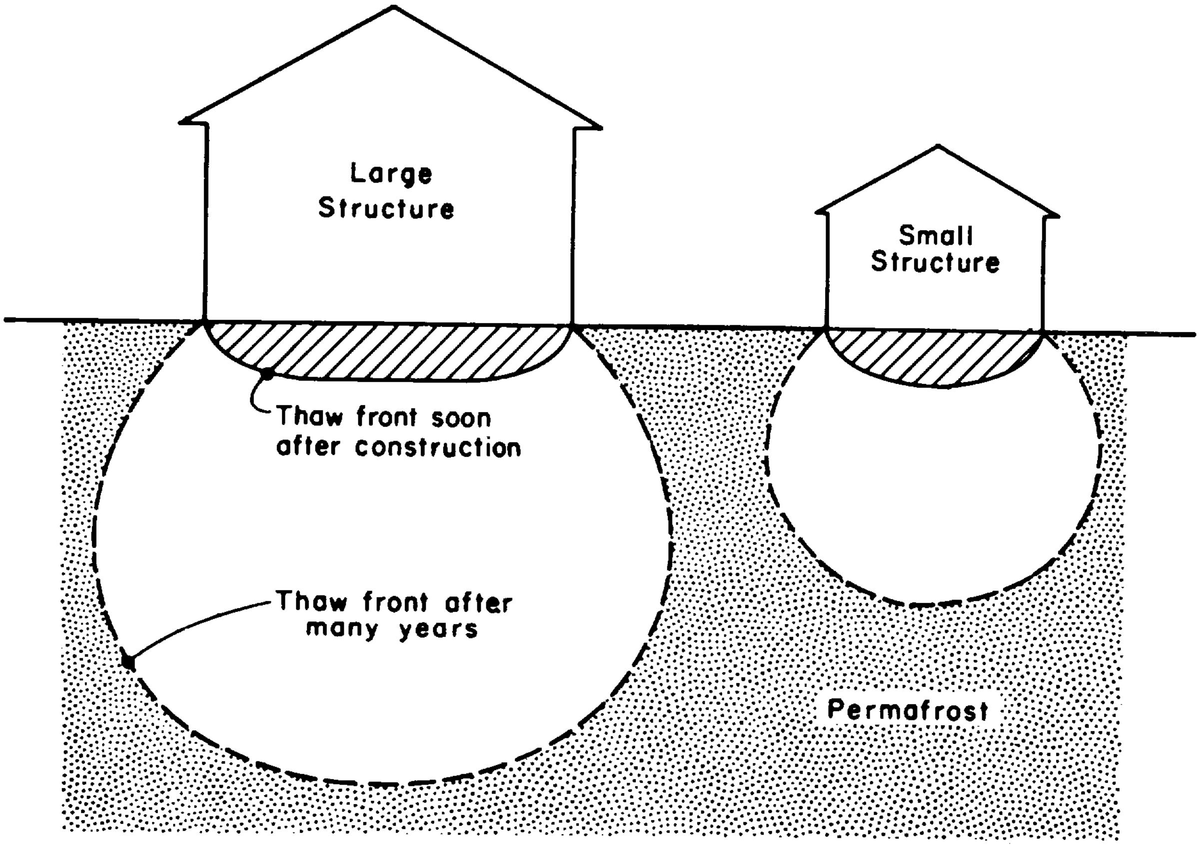


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Figure 4-9. Effect of heated structure size on depth and rate of thaw.

building floor temperature and 32 °F.

(a) Example: Estimate the depth of thaw after a period of one year for a building floor consisting of 8 inches of concrete, 4 inches of cellular glass insulation, and 6 inches of concrete, placed directly on a 5-feetthick sand pad overlying permanently frozen silt for the following conditions:

Mean annual temperature (MAT), 20 F.

Building floor temperature, 65 °F.

Sand pad: Dry unit weight γ_d = 72 lb/ft^3, w = 45 percent.

Concrete: Coefficient of thermal conductivity, K = 1.0 Btu/ft hr °F; Volumetric heat capacity, C = 30 Btu/ft^3 °F.

Insulation: K = 0.033 Btu/ft hr °F, C = 1.5 Btu/ft^3 °F.

The resistances of the three floor layers are in series, and the floor resistance Rf is the sum of the three layer resistances (d = thickness of layer in feet).

$$R_v = \frac{d}{k} = \frac{8}{(12)(1.0)} + \frac{4}{(12)(0.033)} + \frac{6}{(12)(1.0)}$$
$$= 11.2 \text{ ft}^2 \text{ hr °F/Btu}$$

The average volumetric heat capacity of the floor system is

$$C_f \quad \frac{(30)(8) + (1.5)(4) + (30)(6)}{8+4+6}$$
$$= 23.7 \text{ Btu/ft}^3 \text{ °F}$$

The solution to this problem (table 4-2) predicts a total thaw depth of 7.8 feet. This solution did not consider edge effects; i.e., a long narrow building will have less depth of thaw than a square building with the same floor area because of the difference in lateral heat flow.

(b) Figures 4-3, 4-10, 4-11 and 4-12a show measured rates of thaw beneath buildings placed direct-

Table 4-2. Thaw Penetration Beneath a Slab-on-Grade Building Constructed on Permafrost[14]

Layer	γ_d	w	d	Σd	C	K	L	Ld	ΣLd	$\overline{L}$	Cd	ΣCd	$\overline{C}$	μ	λ	λ^2	R_n	ΣR	$\Sigma R + \frac{R_n}{2}$	nI	ΣnI
Floor	--	--	1.5	1.5	24	--	0	0	0	0	36	--	--	--	--	--	11.20	0	5.60	--	--
Sand	133	5.0	5.0	6.5	28	1.54	960	4,800	4,800	738	140	176	27	1.21	0.68	0.463	3.25	11.20	12.82	5,540	5,540
Silt a	72	45.0	1.5	8.0	37	0.90	4,650	6,970	11,770	1,470	55	231	29	0.65	0.77	0.593	1.67	14.45	15.29	7,480	13,020
Silt b	72	45.0	1.3	7.8	37	0.90	4,650	6,050	10,850	1,390	48	224	29	0.69	0.765	0.586	1.44	14.45	15.17	6,520	12,060

Where: L = volumetric latent heat of fusion = $144\ \gamma_d \frac{w}{100}$ (see Figure 8, TM5-852-6[14])

$\overline{L} = \Sigma Ld/\Sigma d$

$\overline{C} = \Sigma Cd/\Sigma d$

$\mu = V_s\left(\frac{\overline{C}}{\overline{L}}\right)$ and $V_s = 65-32=33$

λ determined from Figure 13, TM5-852-6, using μ and α

$\alpha = \frac{V_o}{V_s}$, V_o = initial temperature differential = 32-M.A.T. = 12

$\alpha = \frac{12}{33} = 0.364$

$nI = \frac{Ld}{24\lambda^2}\left(\Sigma R + \frac{R_n}{2}\right)$

Computations for Thaw Penetration:

Surface thawing index (nI) = 33 x 365 = 12,050 degree-days

$$nI\ (\text{Sand}) = \frac{4,800}{24(0.463)}(12.82) = 5,540 \text{ degree-days}$$

$$nI\ (\text{Silt a}) = \frac{6,970}{24(0.593)}(15.29) = 7,480 \text{ degree-days}$$

$$nI\ (\text{Silt b}) = \frac{6,050}{24(0.586)}(15.17) = 6,520 \text{ degree-days}$$

Total Thaw Penetration = 7.8 ft.

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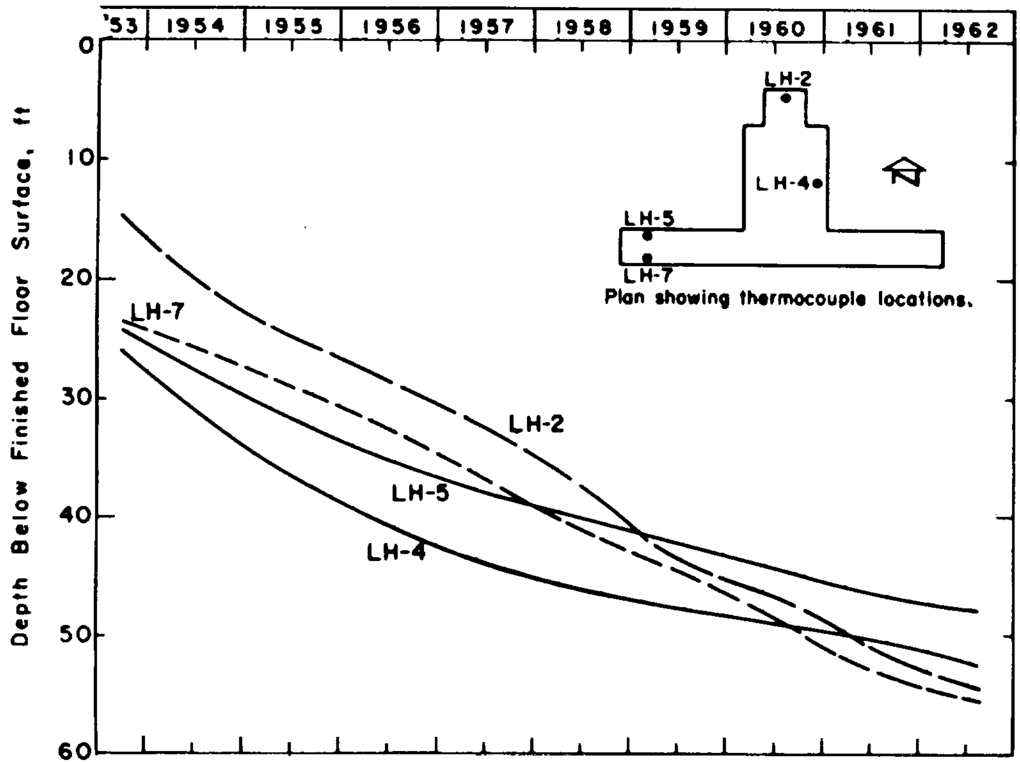

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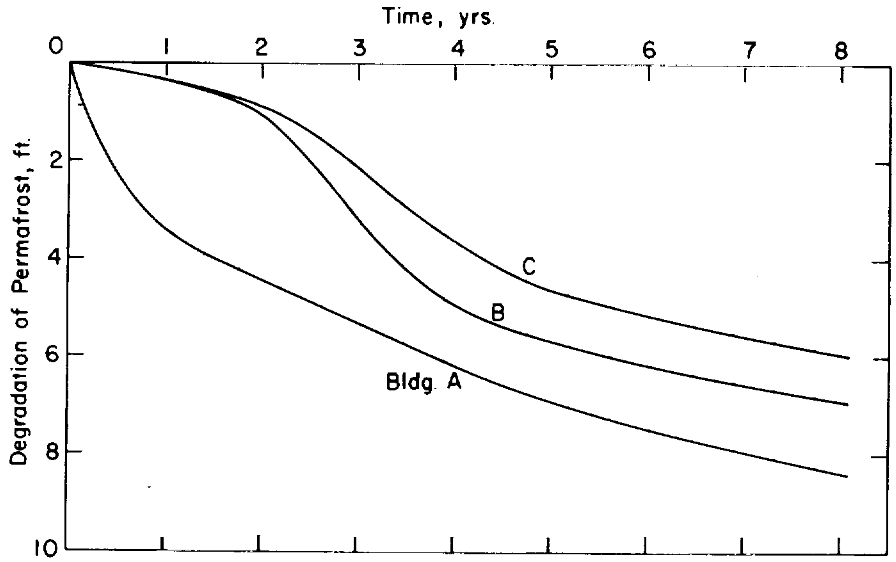
Figure 4-10. Degradation of permfrost under five-story reinforced concrete structure, Fairbanks, Alaska. Foundation: perimeter wall footing wallfooting and interior spreadfootings; uninsulated basement floor 3 to 6 feet below ground surface. Soil types: sandy gravels to silty sands.



		Bldg A	Bldg B	Bldg C
Foundation	Floor	4 in. concrete slab	Wood floor with 4 in. batt insul.	Wood floor with 4 in. batt insul.
	Mat	4 ft gravel	2 ft gravel	2 ft gravel over 6 in. cell concrete over 2 ft gravel
Conditions at end of construction	Thaw depth in silt	4.6 ft	2.9 ft	3.5 ft
	Depth to permafrost (from top of gravel fill)	8.6 ft	4.9 ft	8.0 ft

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Figure 4-11. Permafrost degradation under 16-feet-square heated test buildings without air space, beginning at end of construction. Table shows foundation conditions. See figure 4-19 for site conditions.

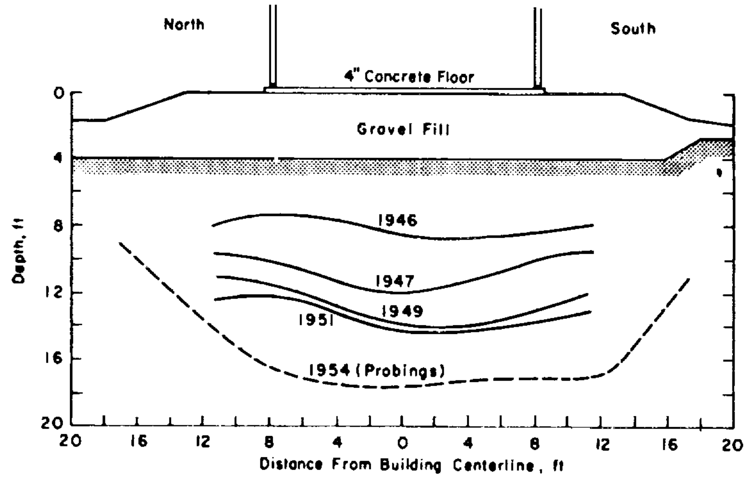

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Figure 4-12a. Typical foundation thaw near Fairbanks, Alaska. See figure 4-19 for "Site Conditions."

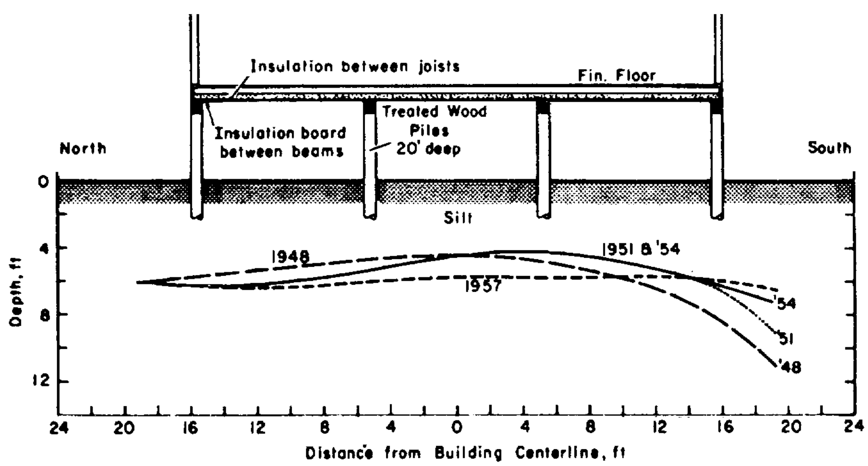

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Figure 4-12b. Typical foundation thaw near Fairbanks, Alaska (Permafrost Degradation Under heated building (32 x 32ft) supported by piles over airspace.)

ly on the ground, over permafrost. Figures 4-3 and 4-10 show data for large reinforced concrete structures, 3 stories and 5 stories, respectively, erected on clean, granular frozen soils which did not contain ice in such form as to cause significant settlement on thawing. Figure 4-11 shows the effects of various combinations of insulation and granular mat on thawing beneath small experimental buildings supported over frozen silt containing much ice. Insulation held back degradation initially but had little effect later. Figure 4-12 compares the continuing degradation under a small building without foundation ventilation with the thermal stability achieved by supporting a structure on piles with an airspace.

(3) *Ventilated foundations.* The most widely employed, effective and economical means of maintaining a stable thermal regime in permafrost under a heated structure is by use of a ventilated foundation. In such a foundation, provision is made for either open or ducted circulation of cold winter air between the insulated floor and the underlying ground. The air circulation serves to carry away heat both from the foundation and from the overlying building, freezing back the upper layers of soil which were thawed in the preceding summer.

(a) Cold air passing through a simple air space beneath a building or through a ducted foundation ventilation system is gradually warmed, reaching the outlet side with a reduced air freezing index. Thus, freezeback in a ventilated foundation tends to progress from the intake toward the outlet side, as indicated by the asymmetrical curve of thaw penetration depth in figure 4-28. In summer also, thaw tends to occur progressively across the foundation in the direction of air flow. The freezing index at the outlet must be sufficient to counteract the thawing index at that point in order to insure annual freezeback of foundation soils. In borderline discontinuous permafrost areas, this freezeback is more difficult to achieve than in colder climates and in these areas it may set a positive limit on the feasible width of buildings for a given type of ventilated foundation design. Even under calm conditions, air circulation will be induced by heating of the air below a building from both the ground and the building. Stacks or chimneys may be used where appropriate to induce increased circulation and they may be found to be a positive requirement. The stack or chimney height and the floor insulation are both very important variables in the foundation design. Increasing insulation thickness will permit lowering the stack or chimney height for the same insulation; increase in stack height will increase the air flow. Potential permafrost degradation problems from flow of ground water in the annual zone must be carefully investigated[200].

(b) The simplest means of implementing foundation ventilation is by providing an open air space under the entire building, with the structure supported on piling in permafrost, columns supported on footings in permafrost, or posts and pads over a gravel layer, as illustrated by figures 4-13 through 4-23. For structures whose narrow width is not more than 20 feet, the air space should technically not be less than 18 inches and where the narrow building width is between 20 to 50 feet not less than 30 inches. However, since access to the air space may be required for foundation adjustments such as jacking or shimming, for inspection or repair of utilities, or for other reasons, the actual depth of the space should be enough so that it may serve as a crawl space, nominally 30 to 36 inches minimum, regardless of size of structure. Beams, sills and other supporting members may occupy part of this space provided all parts of the foundation are accessible for maintenance, free paths of air circulation across the width of the foundation are maintained, and paths for direct conduction of heat from the building into the foundation are kept minimal. In areas subject to snow drifting, too little clearance, excessive numbers of piles or excessive depth of framing members will reduce air velocities and permit drifting and snow accumulation under buildings. For very wide structures or where access to the air space is restricted, induced air circulation by the use of plenums and/or stacks or chimneys, or, less likely, by use of fans, may be required.

(c) When large buildings with heavy floor loads, such as hangars, garages and warehouses, make provision of an open air space difficult, use of ventilation ducts below the insulated floor should be considered. Examples of such designs are shown in figures 4-24 through 4-28. Thermal calculation procedures for ducted foundations are outlined in TM 5-852-/AFM 88-19, Chapter 6 [14]. Ducted foundations are normally much more expensive than open air-space foundations, because of the relatively large volume of concrete and numbers of construction steps involved and because of the cost of pans, pipes, plywood or other special duct-forming items left in place when these materials are used. The design shown in figure 4-24, which makes extensive use of simple prefabricated members, demonstrates an effort to reduce the cost of ducted foundations. Because of the susceptibility to damage of ducted type construction from vertical movements, special care must be taken that the underlying gravel mat is of adequate thickness so that freeze and thaw will remain within non-frost-susceptible materials to eliminate seasonal heave and settlement.

(d) Ventilated foundation design should incorporate a safety factor which provides for complete freezeback of the underlying soil 30 days before the end of the freezing season, using the minimum site freezing index and allowing for any greater freezeback requirement which may exist at the perimeter of the founda-

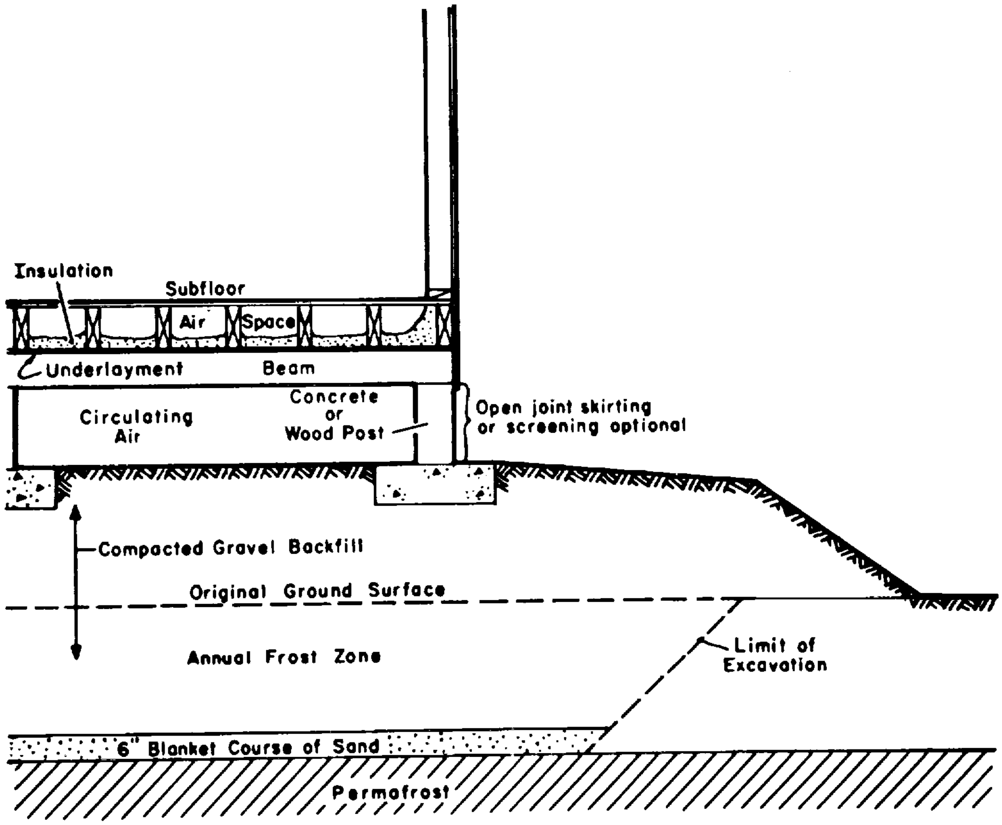

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Figure 4-13. Typical design for light structure with air space and gravel mat (by CRREL).

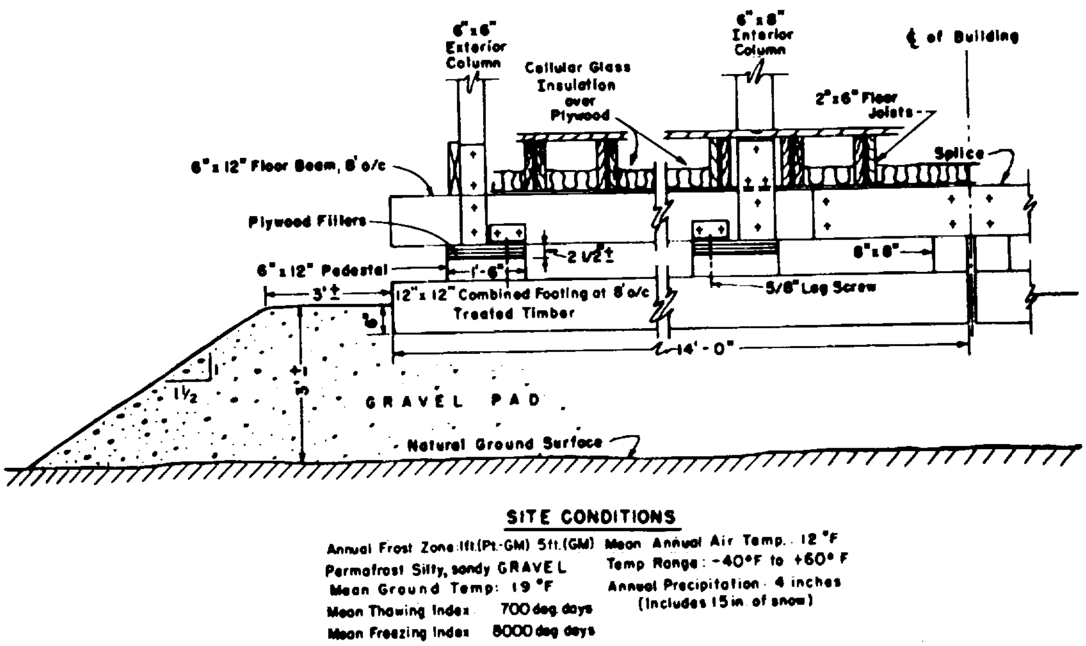

Figure 4-14. Foundation in permafrost area for men's barracks, Thule, Greenland[86].

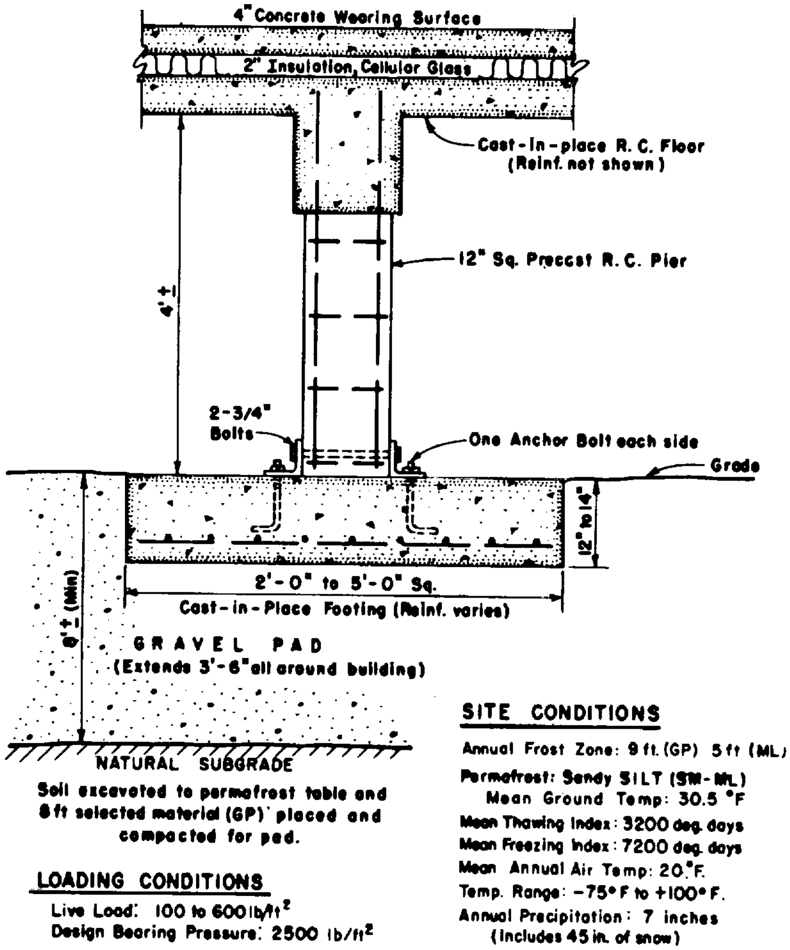

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Figure 4-15. Post and pad type foundation for composite building, Fort Yukon, Alaska[86].

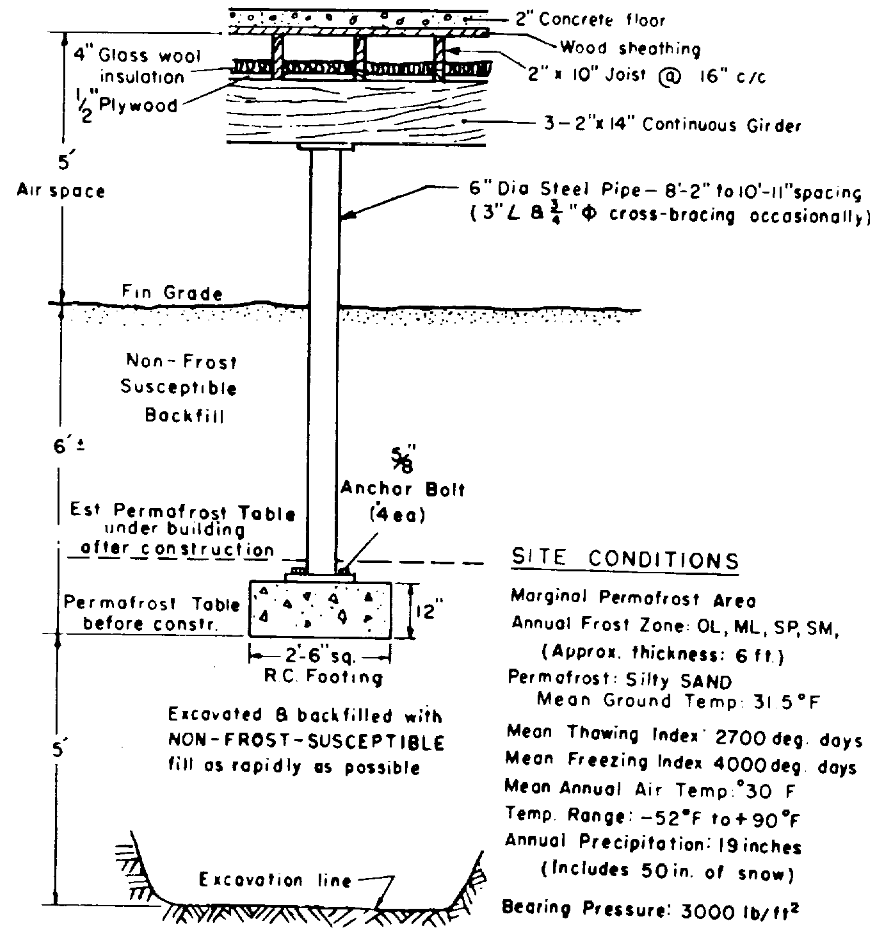

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Figure 4-16. Footing on permafrost foundation, Bethel, Alaska[86].

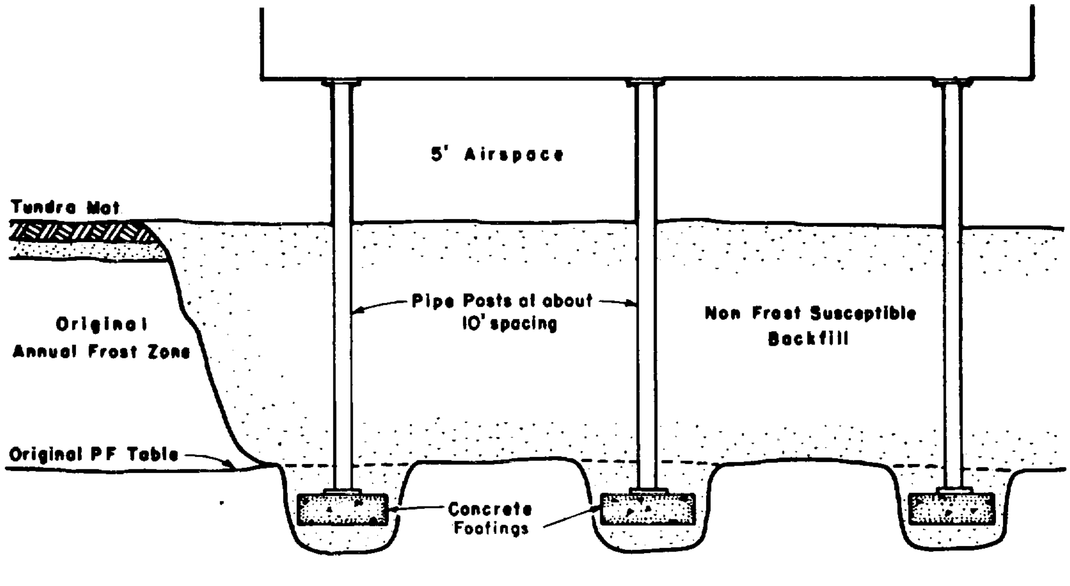

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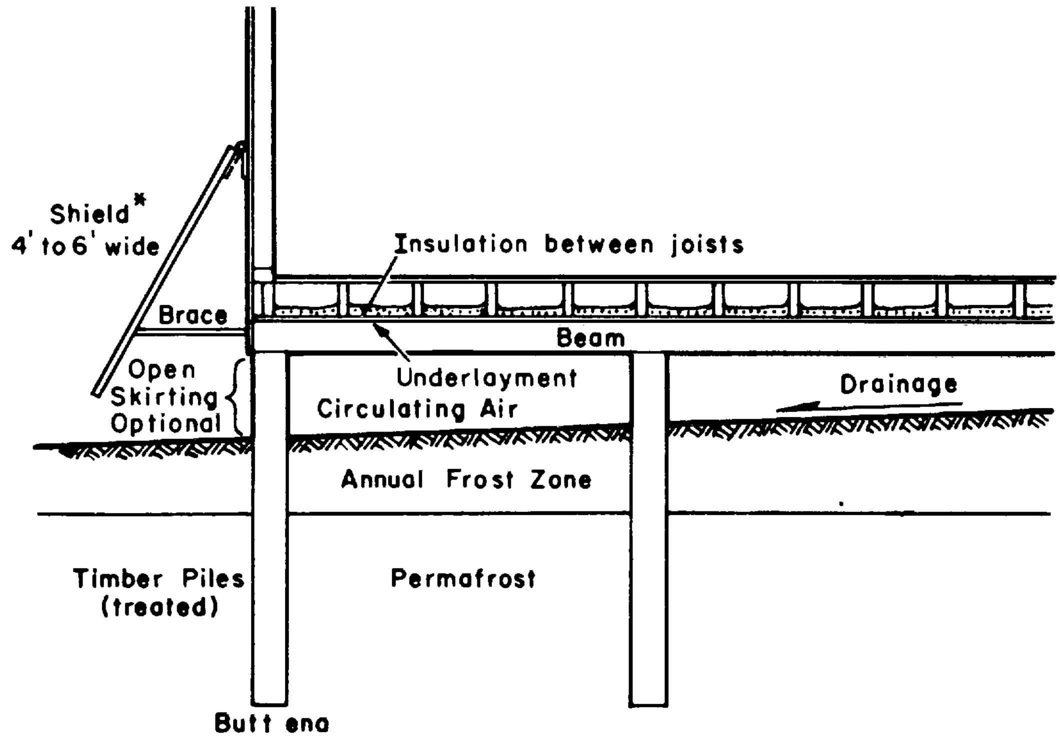
Figure 4-17. Footings on permafrost.



*When necessary, detachable hinged, vented shield may be placed on building walls with southerly to westerly exposures to shade foundation perimeter during summer seasons.

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Figure 4-18. Typical ventilated foundation design for structure supported on piles.

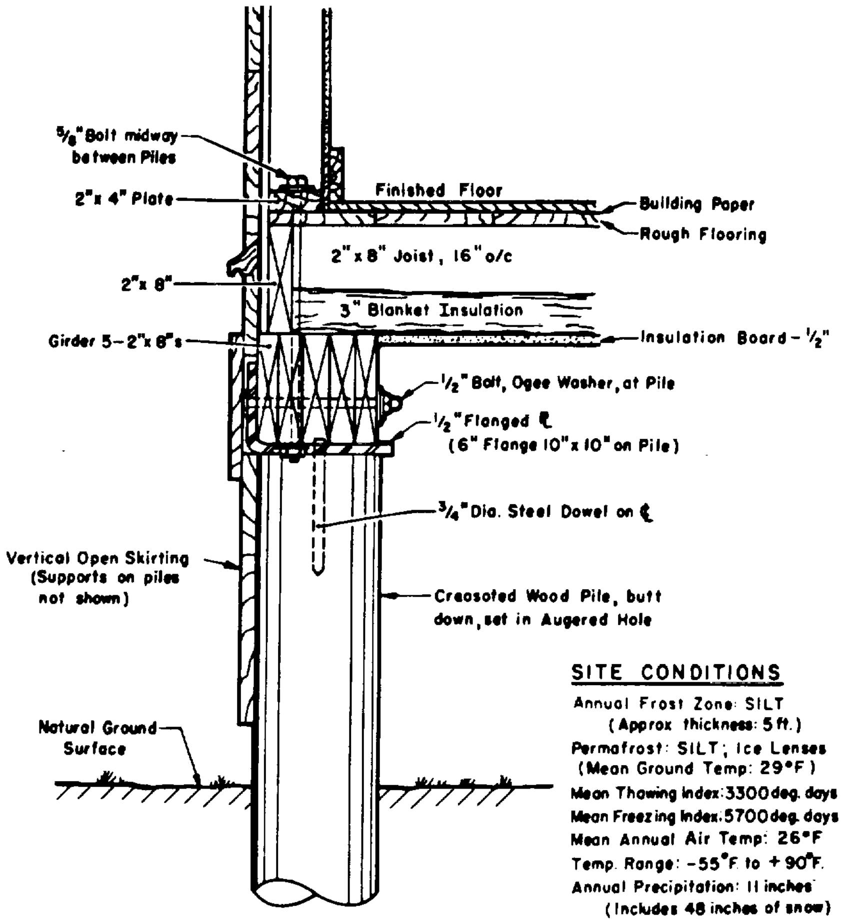

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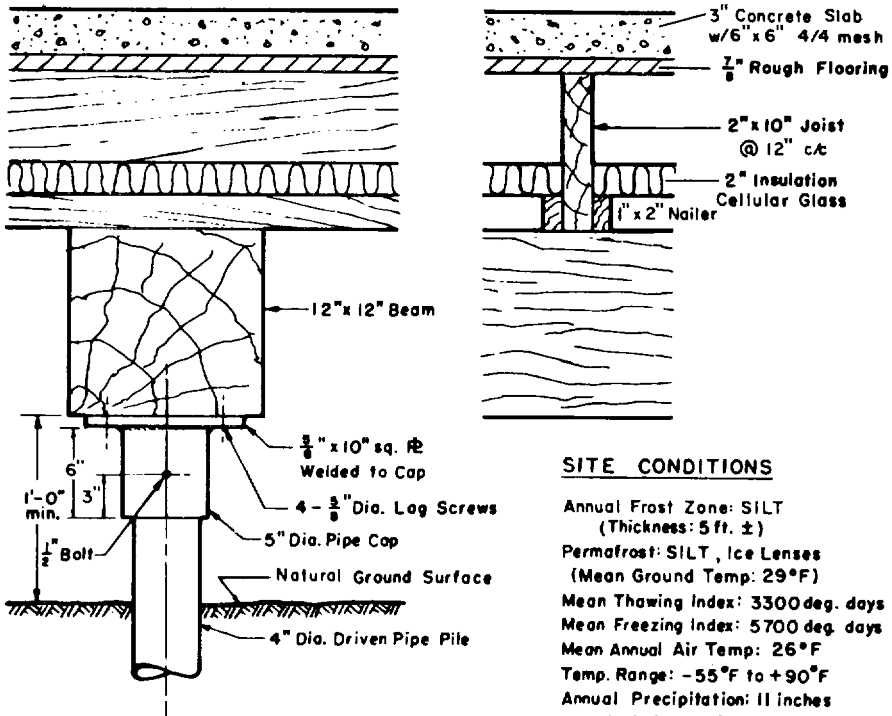
Figure 4-19. Woodpile foundation for small residences, Fairbanks, Alaska.



SITE CONDITIONS

Annual Frost Zone: SILT
(Thickness: 5 ft. ±)
Permafrost: SILT, Ice Lenses
(Mean Ground Temp: 29°F)
Mean Thawing Index: 3300 deg. days
Mean Freezing Index: 5700 deg. days
Mean Annual Air Temp: 26°F
Temp. Range: -55°F to +90°F
Annual Precipitation: 11 inches
(Includes 48 in. of snow)
Low Wind

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Figure 4-20. Typical pile foundation for light utility building, Fairbanks, Alaska.

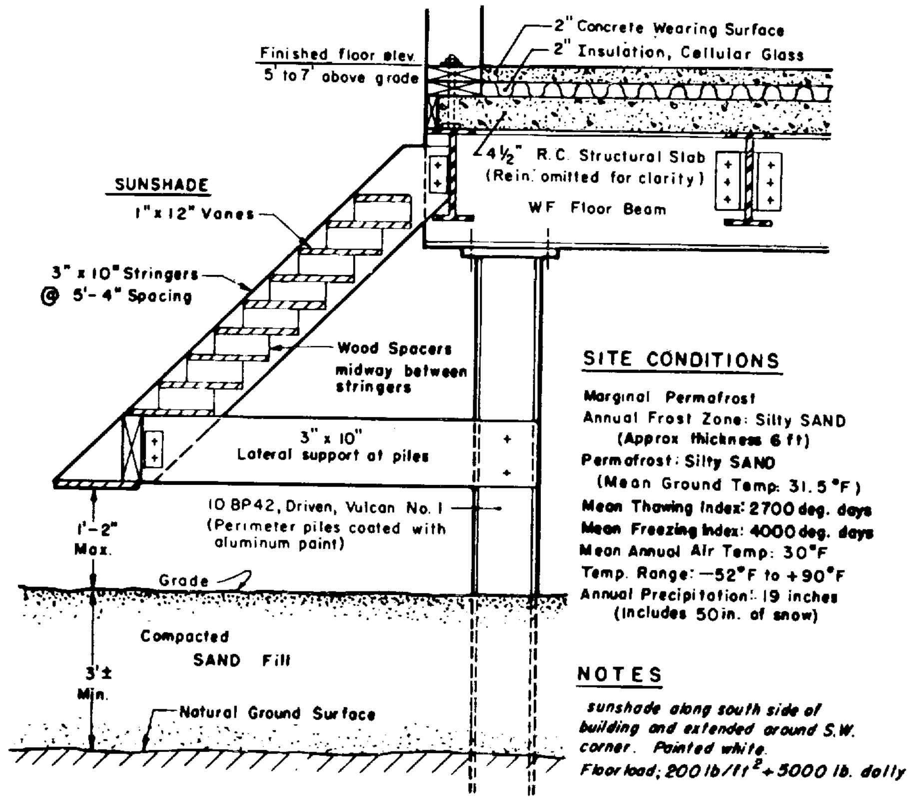

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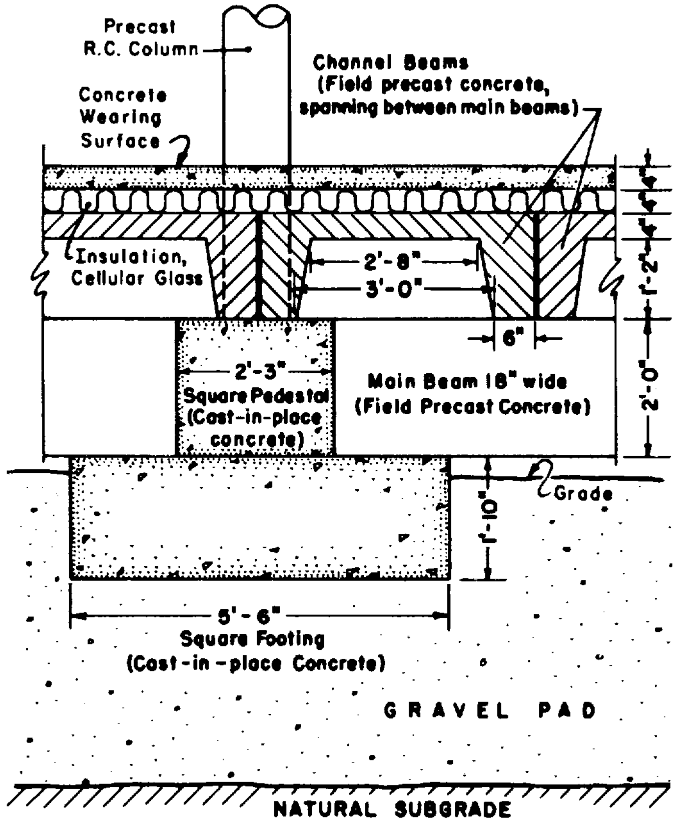
Figure 4-21. Steel pile foundation for utility building showing sunshade, Bethel, Alaska.



Steel reinforcement in concrete omitted for clarity

SITE CONDITIONS

Annual Frost Zone: 1ft.(Pt-GM) 5ft.(GM)
Permafrost: Silty, sandy GRAVEL
Mean Ground Temp: 19°F
Mean Thawing Index: 700 deg. days
Mean Freezing Index: 8000 deg. days

Mean Annual Air Temp: 12°F
Temp. Range: -40°F to +60°F
Annual Precipitation: 4 inches
(Includes 15 in of snow)

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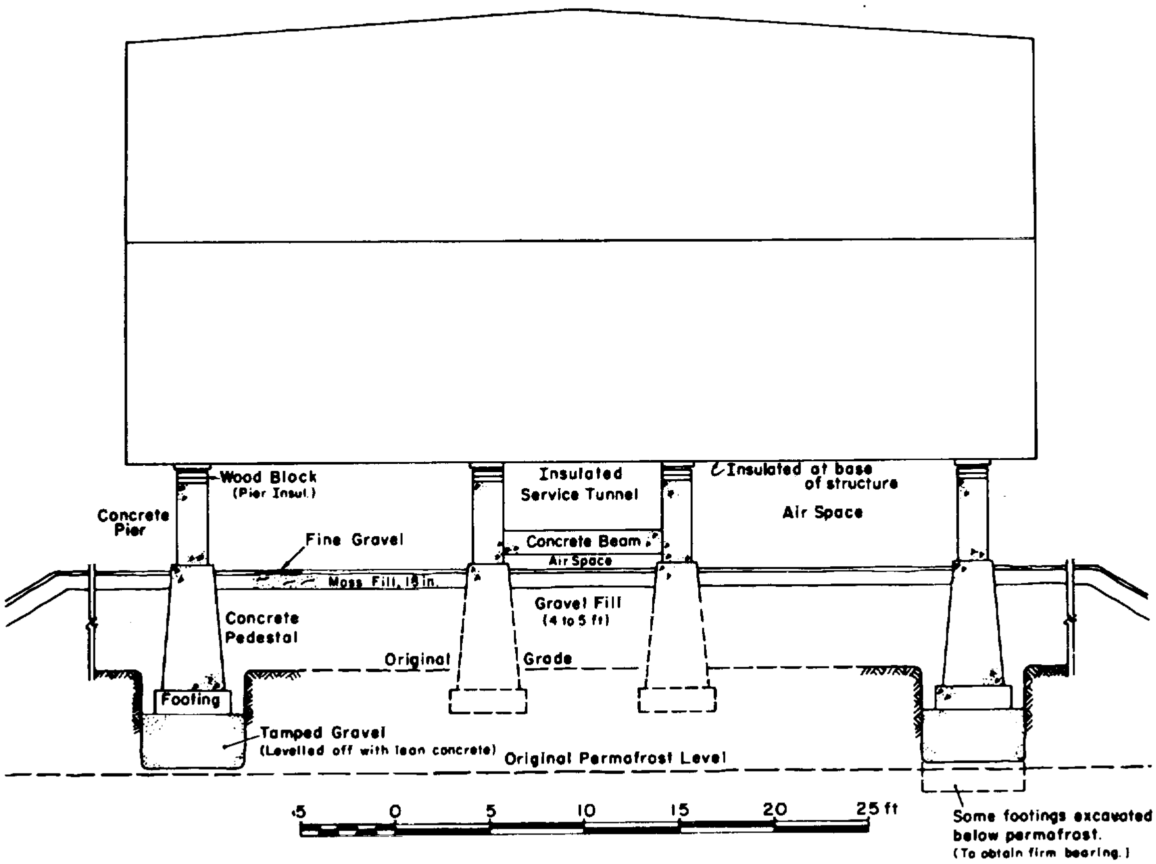
Figure 4-22. Foundation for men's club, Thule, Greenland.



(Courtsey of American Society of Civil Engineers)

(Courtesy of American Society of Civil Engineers)

Figure 4-23. Two story steel frame building on footings and piers at Churchill, Manitoba, Canada[138]. Subgrade is sand and silt interspersed with gravel and large boulders.

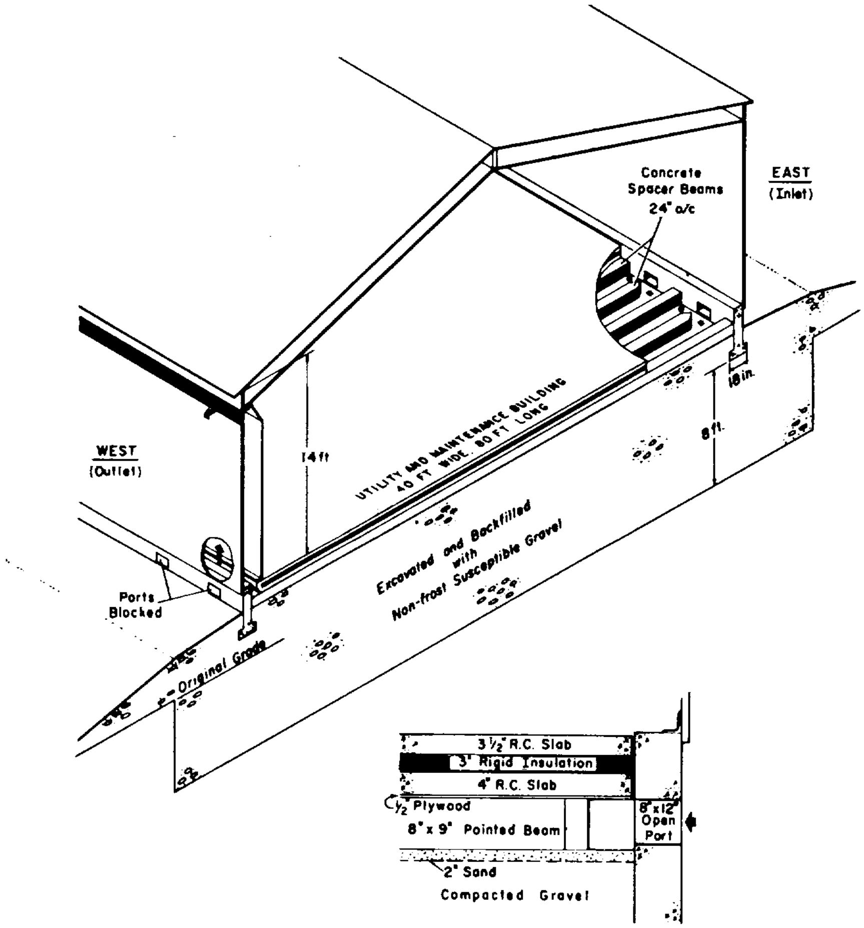


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Figure 4-24. Utility and maintenance building, Fairbanks, Alaska (by CRREL). See figure 4-25 for site conditions.

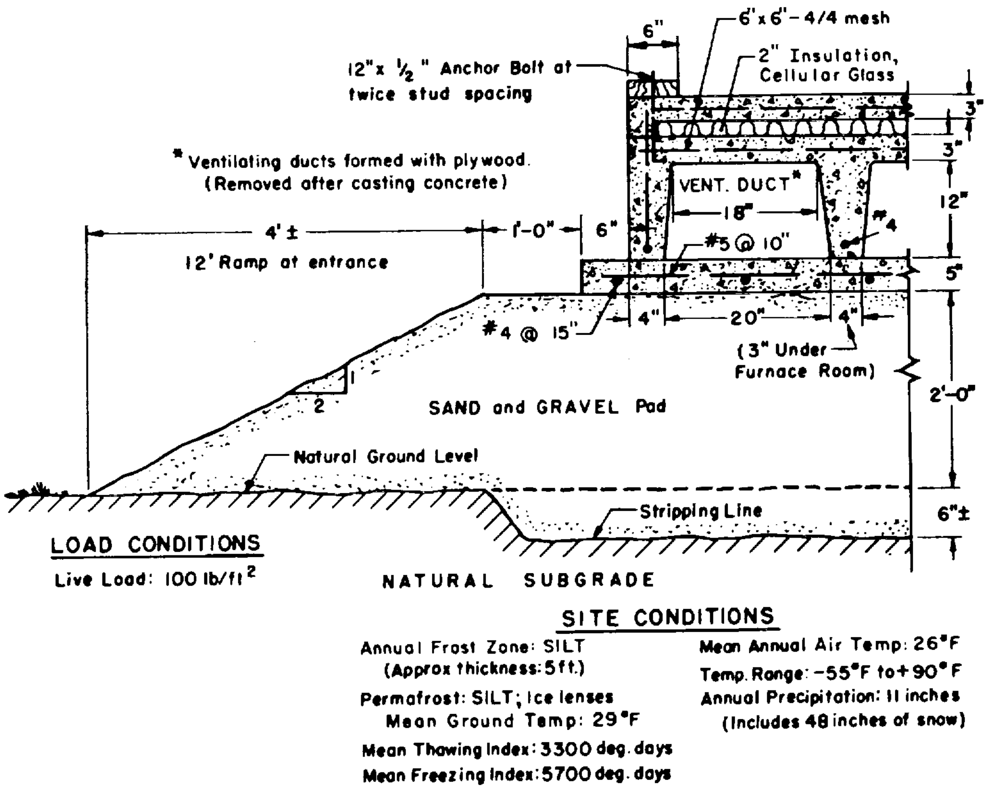

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Figure 4-25. Ducted foundation for garage, Fairbanks. Alaska[86].

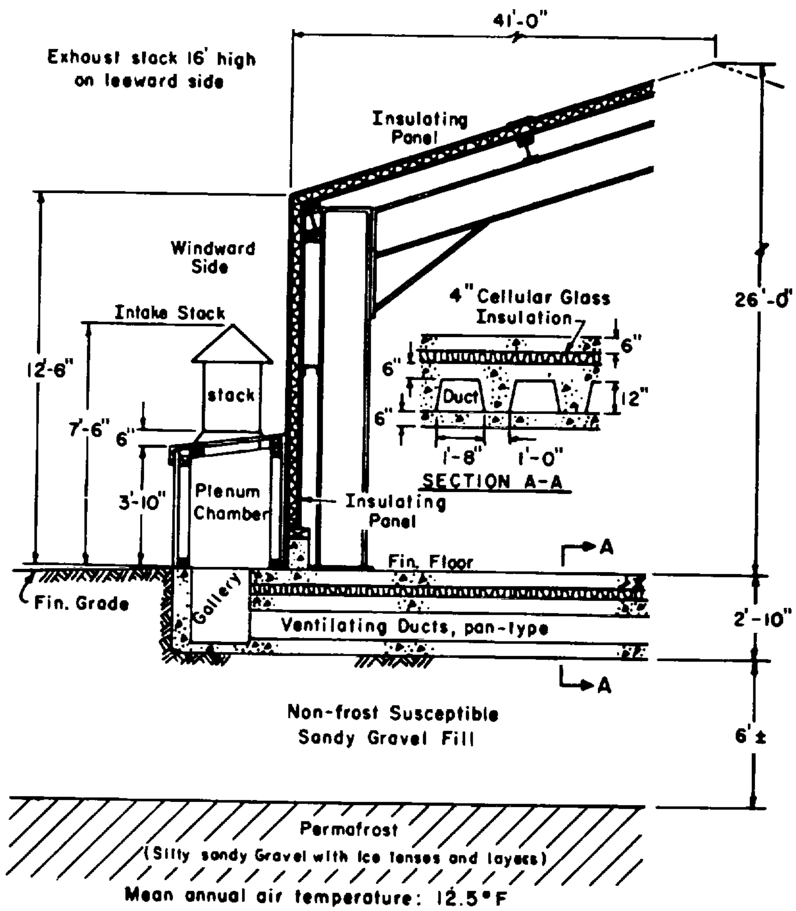

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Figure 4-26. Pan duct foundation, Thule, Greenland[86]. See figure 4-22 for site conditions.

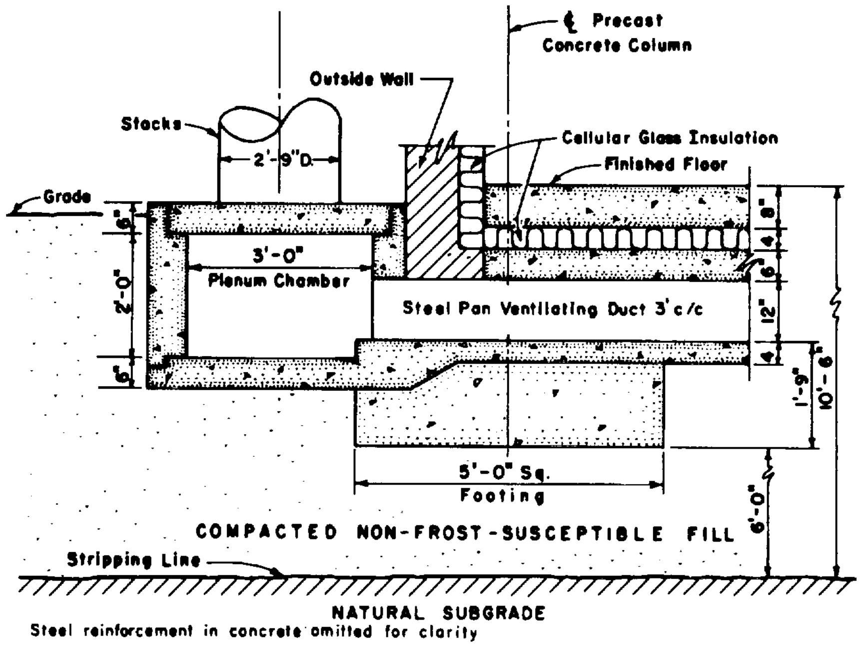

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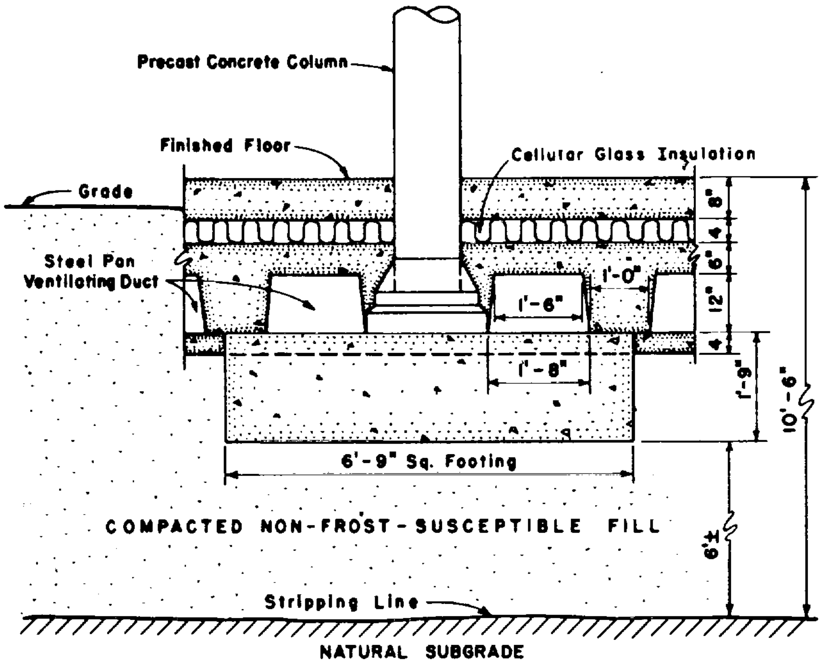
Figure 4-27a. Typical Pan Duct Foundation, Showing Section at Plenum chamber and at Interior Column for Warehouse, Sondrestrom AB, Greenland (Section at Plenum Chamber)



Steel reinforcement in concrete omitted for clarity

SITE CONDITIONS
Annual Frost Zone SILT to silty SAND
(Approx. thickness: 8 ft.)
Permafrost GRAVEL through SAND to SILT
Mean Ground Temp. 27°F
Mean Thawing Index. t900 deg. days
Mean Freezing Index 5100 deg. days
Mean Annual Air Temp. 24°F
Temp Range -49°F to +73°· F
Annual Precipitation 5 inches

LOADING CONDITIONS
Live LOAD. 250 lb/ft^2
Design Footing Bearing Pressure: 4000 Ib/ft^2

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Figure 4-27b. Typical Pan Duct Foundation, Showing Section at Plenum Chamber and at Interior Column for Warehouse, Sondrestrom AB, Greenland (Section at Interior column)

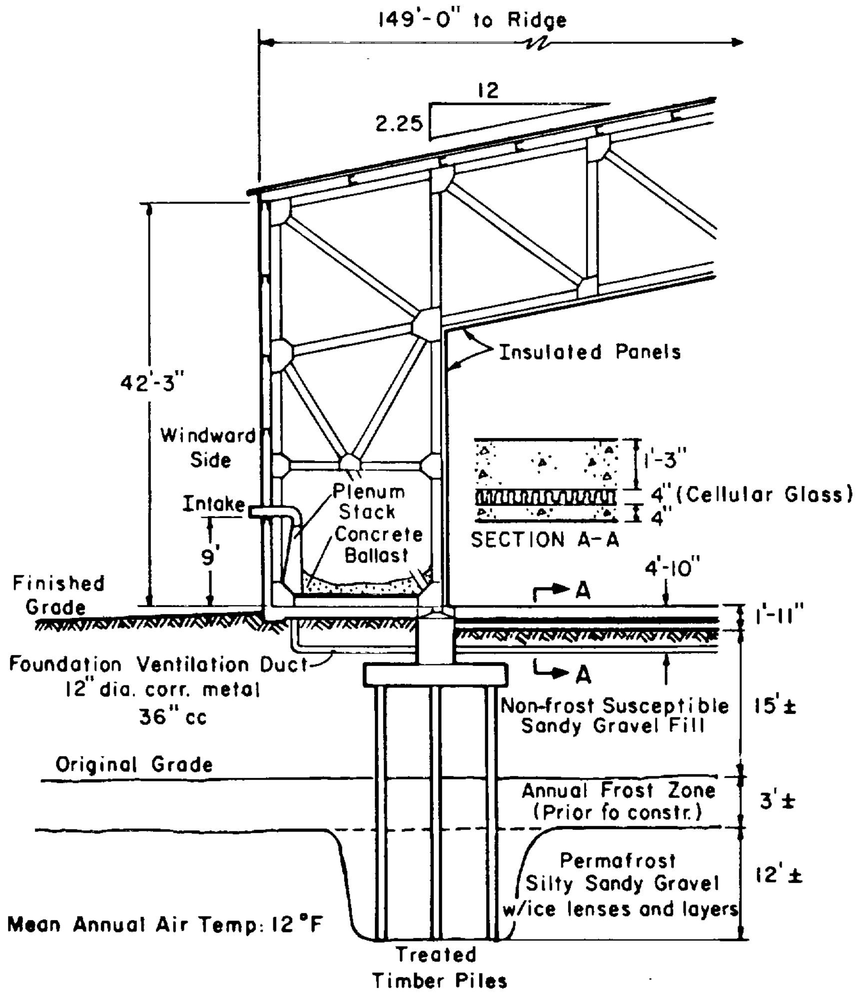

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Figure 4-28a. Foundation Details and Maximum Thaw Penetration for Selected Years, Hangar at Thule, Greenland (Deep air duct foundation details. Forced circulation by fans may be required where natural draft is not sufficient. Exhaust for cooling duct is located on leeward side, 32 feet above grade. One plenum stack for each six or seven ducts. Arch tie rod omitted for clarity.)

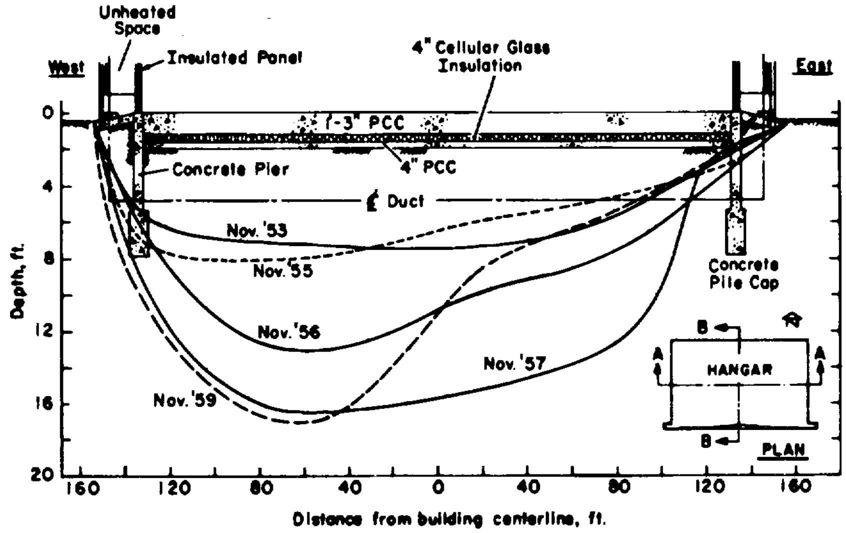

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Figure 4-28b. Foundation details and maximum thaw penetration for selected years, hangar at Thule, Greenland (Longitudinal section A-A parallel to 12-inches-diameter corrugated metal cooling ducts)

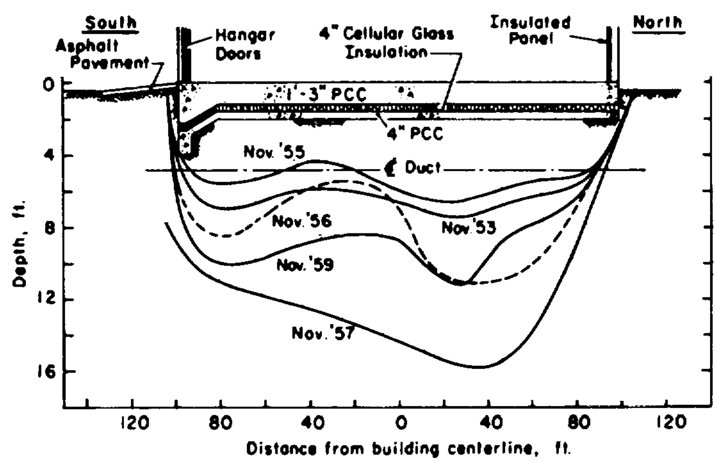

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Figure 4-28c. Foundation details and maximum thaw penetration for selected years, hangar at Thule, Greenland. (Transverse section B-B perpendicular to 12-inches-diameter corrugated metal cooling ducts) tion. Since only about 5 percent of the freezing index is usually accumulated in the last 30 days of the freezing season, this is a very modest factor of safety. At one subarctic site, the complete freezeback of the soil on the downwind side of a building with ducted foundation was not completed until after the soil immediately under the foundation slab commenced its summer thaw. A slight increase in the building's interior operating temperature could have serious consequences under such a situation, not only because of the risk of permafrost degradation but also because of the possible lowering of pile supporting capacity if the structure is pile-supported.

(e) Experience has shown that it is desirable that ventilated foundations be sufficiently elevated, positioned and sloped relative to the surrounding terrain to avoid accumulation of surface water, to drain away in summer thaw water from any accumulations of ice and snow from the preceding winter and to prevent lateral migration of water through the annual thaw zone. Figure 4-25 illustrates an elevated ducted foundation. Such ducts are also more immune to blocking with soil accumulations.

(f) Ducts depressed below the ground surface as in figures 4-26, 4-27 and 4-28 are likely to collect water from the ground or ice and soil from snow and dust infiltration, which restrict or block air flow through the ducts. If ground water rises to the duct level, soil may also be piped into the ducts. Blockage is often unnoticed until after water in the ducts has frozen. Such obstructions are very difficult to remove. Steam thawing may be required to open them; this is not only somewhat complicated but may also cause thermal damage unless carefully controlled. Condensation of ice crystals in the ducts from moist air may also block the ventilating ducts if they are kept in operation when air temperatures become higher than the temperatures of the duct walls in the spring. Tobiasson[200] has pointed out that for below-grade duct systems, manifolds and perhaps the ducts themselves should be large enough to permit entry of maintenance personnel for inspection and removal of blockage, and provisions should be incorporated to minimize the amount of snow infiltration and to remove any material which does enter. Experience indicates that when plenum and/or stacks or chimneys, as illustrated in figures 4-24, 4-26, 4-27 and 4-28, are needed to increase air flow by the stack effect or to raise intakes and outlets sufficiently to be above maximum snow accumulation levels, a chimney space incorporated so as to take advantage of the building heat, as in figure 4-24, is preferable to independent exterior exhaust stacks in which cooling of the rising air tends to diminish the draft.[200] Insulation of the stacks can reduce this difference. Systems should be free of air leaks to insure maximum circulation effectiveness.

(g) Blower systems may be used when conditions require increased volume of air circulation in ducts, but at the expense of increased mechanical complexity, increased operating costs, and necessity for alertness to make sure the system is turned on and off and the air flow controls set correctly at proper times (see further discussion of this point later in this chapter).

(h) Part or all of the air space of a ventilated foundation has sometimes been used for unheated storage purposes, particularly when extra height of air space has resulted from variations of the natural topography. However, air circulation at the ground and foundation freezeback are easily impaired by such storage, and extra accumulation of snow may be induced.

(i) In ducted foundations of the general types shown in figures 4-25 and 4-27, the vertical concrete sections between individual ducts should be kept relatively thin in order to minimize conduction of heat through these members directly to the foundation. In pile foundations, conduction of heat into the ground by the piles should be minimized by techniques described in *f* below.

(j) Experience has shown that where blowing or drifting snow occurs in winter it is very important to align and locate the structure so as to minimize snow drifting which may in any way affect the structure. Unless ventilation openings of foundations are placed and oriented so that they will not become blocked by snow, the snow drifting may restrict or prevent necessary seasonal freezeback of the foundation. Size and shape of structures and position with respect to prevailing wind and to other structures, to fences and vegetation affecting wind flow, and to adjacent snow removal operations are very important in determining snow drift patterns. Everything else being equal, maximum drifting tends to occur on windward and lee sides of obstructions. However, even if access of winter air to the foundation is completely shut off by snow on the windward and lee sides, ventilating action of an open air space type foundation may still be satisfactory if the other two sides remain open and so long as drifting of snow into open space under the building itself remains insufficient to significantly insulate the foundation materials against freezeback. Open air space type foundations subject to drifting therefore should be designed and oriented to depend on air flow through the foundation at right angles to the wind direction; the shortest dimension of the foundation should then be at right angles to the prevailing wind. Provisions should also be made against other possible problems such as blocking of ground level ventilation intakes or outlets by accumulation of snow next to the foundation from roof discharge or from snow plowing operations. If snow blockage problems cannot be practically avoided through adjustment of orientation and location, use of flatter roof pitch or greater overhang, or other means, it may be necessary to employ plenum chambers and stacks or chimneys as shown in figures 4-26, 4-27 and 4-28. If other considerations should make it essential to rely on ventilation flow parallel with the predominant wind direction and it is not feasible to elevate the intake and exhaust sufficiently, fences or shrubbery may be installed upwind of the structure to induce drifting at that position and thus to minimize drifting close to the building itself. Sometimes snow ridges pushed by snow plowing operations may be counted on to induce drifting in desired locations. Extreme distance of drifting behind a snow fence is about 25 times the height of fence. Principles of snow drift control have been outlined by Mellor[77]. The use of completely closed skirting around foundations constructed with open air spaces must be avoided. Open picket-type skirting around foundations has been successfully used to permit ventilation, while preventing significant snowdrifting under the building, keeping children and animals out of the air space, and beneficially modifying the overall architectural appearance of the building. If used, such skirting should be elevated sufficiently above the ground to avoid damage due to frost heaving. Wire mesh may also be employed over openings to foundation air spaces but the mesh openings should not be smaller than about 2 inches.

(k) For maximum effectiveness a foundation cooling system utilizing natural low winter temperatures should be shut off in the spring when air temperatures reach such a level that circulation of the ambient air through the system would add to the summer heat input into the foundation. Turning off the system in the spring and turning it on again in the fall is necessary for systems using forced circulation, or stack or chimney systems. However, experience shows that when such a system is dependent upon manual opening or closing of ports or dampers, or turning electrical switches on or off, and these operations are required only twice a year, the necessary actions may be forgotten or may be carried out incorrectly. Experience also shows that system operating manuals are easily misplaced. Therefore, whenever possible, designs should be selected which are automatic in operation and do not require specific manual actions. Fail-safe differential thermostat control systems can reduce these problems, but still require dependable power supply, checking for proper operation of controls, and resetting of circuit breakers. Reliability becomes less in relatively complex systems which involve numerous dampers, blowers or other elements. Therefore, simple ventilated foundations or through-duct systems of the type illustrated in figure 4-25 which are entirely free of control mechanisms requiring setting are far preferable. Where required, operating directions for dampers, switches, etc. should be stenciled directly adjacent to the particular control element to ensure that the necessary guidance is available when needed. If dampers are installed they should be placed on the upwind side[200]. Eliminating downwind dampers allows convection currents to remove warm air from ducts even with the upwind damper closed. Buildings with ventilated foundations have greater potential rates of heat loss in winter than structures resting directly on the ground because they are exposed to the cold air on all their surfaces. Special care is therefore required in insulation and heating. This is further discussed in d below.

(l) If snow drifting is not a problem or if the intake and exhaust can be elevated so that snow drifting will not interfere with air flow, structure orientation should be such that maximum velocity and effectiveness of air circulation will be obtained, combining both thermal and wind-induced effects. If wind is of significant strength and consistency in direction during the freezing season it is then desirable to orient air intakes into the wind and to configure the exhaust end of the system to maximize wind-induced suction, whether wind-induced draft is taken into account in design computations or not. In many areas, however, this may not be practical because winter winds are too light or too variable in direction and velocity; in such cases possible benefits from wind-induced drafts should be ignored in developing the system design. Where wind-blown precipitation comes from variable directions, rotating wind-oriented ventilator units may be employed at tops of exhaust stacks to minimize snow ingestion and maximize draft. Exhausts should terminate at positions on the lee side of the building.

(m) Thermal analysis of simple ventilated foundation. The depth of thaw under an open air space type ventilated foundation may be approximated from figure 4-4a for certain homogeneous soil and moisture content conditions. For situations not covered by figure 4-4a, the depth of thaw should be calculated by means of the modified Berggren equation for either a homogeneous or multilayered system as applicable, using procedures outlined in TM 5-852-6/AFM 88-19, Chapter 6[14]. An n-factor of 1.0 is applicable for determining the surface thawing index of the shaded area under the building, under either approach.

(n) Thermal analysis of ducted foundation. No simple mathematical expression exists to analyze the heat flow in the case of a ventilated floor system consisting of a duct or pipe system installed within or at some depth beneath the floor, with air circulation induced by stack effect. The depth to which freezing temperatures will penetrate is computed by means of the modified Berggren equation except that the air-freezing index at the outlet governs. The index is influenced by a number of design variables, i.e., average daily air temperatures, inside building temperatures, floor and duct or pipe system design, temperature and velocity of air in the system, and stack height. Cold air passing through the ducts acquires heat from the duct walls and experiences a temperature rise longitudinally along the duct with a reduction in air-freezing index at the outlet. Field observations indicated the inlet air-freezing index to closely approximate the site air-freezing index. The freezing index at the outlet should be sufficient to counteract the thawing index and insure freeze back of foundation soils.

Example: Determine the required thickness of a gravel pad beneath the floor section shown in figure 4-29 to contain all thaw penetration and the required stack height to insure freezeback of the pad on the outlet side of the ducts for the following conditions:

Duct length *l*, 220 feet.
Gravel pad: $\gamma d = 125$ lb/ft^3, w = 2.5 percent.
Outlet mean annual temperature, 32 °F. (This is a conservative assumption.)
Minimum site freezing index, 4,000 degree-days.
Freezing season, 215 days.
Thawing season, 150 days (period during which ducts are closed).

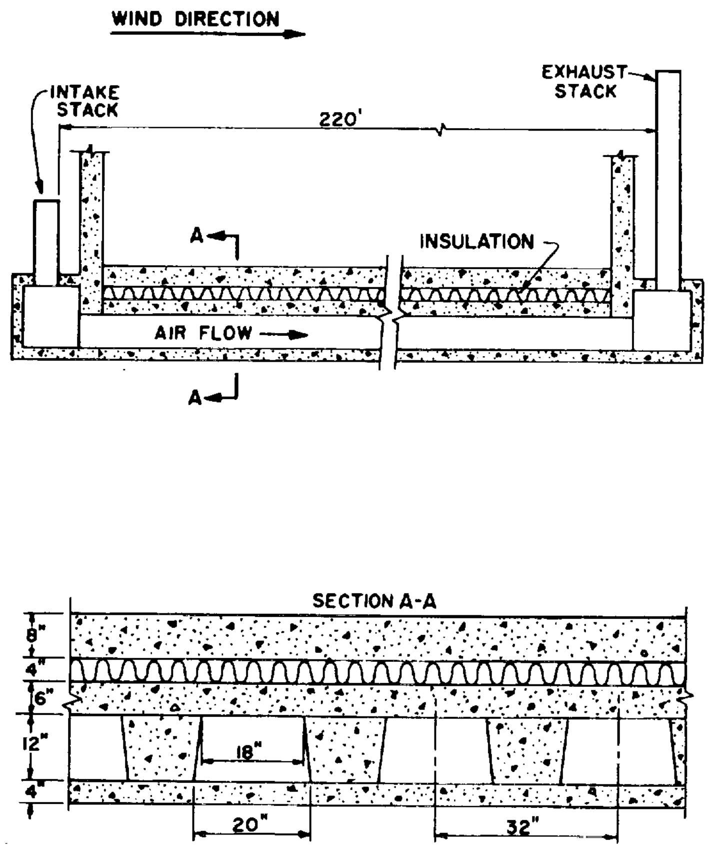

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Figure 4-29. Schematic of ducted foundation[14.]

Building temperature, 60 0F.

Thermal conductivity of concrete, Kc = 1.0 Btu/ft hr°F.

Thermal conductivity of insulation, Ki = 0.033 Btu/ft hr °F.

Other data required for solution are obtainable from TM 5-852-6/AFM 88-19, Chapter 6" or are introduced later. The thickness of gravel pad required is determined by the following equation (derived from the modified Berggren equation):

$$X = KR_f \left(\sqrt{1 + \frac{48\lambda^2 I_f}{KLR_f^2}} - 1 \right)$$

where

K = average thermal conductivity of gravel
$= 1/2\ (K_u + K_f)$
$= 1/2\ (0.7 + 1.0)$
$= 0.85$ Btu/ft hr °F

$*R_f$ = thermal resistance of floor system

$$= \Sigma \frac{\text{thickness layer}}{K}$$

$$= \frac{18}{12\ (1.0)} + \frac{4}{12\ (0.033)} + \frac{12}{12\ (1.0)} = 12.5\ \text{ft}^2\ \text{hr°F/Btu}$$

(In the computations, the dead airspace is assumed equivalent to the thermal resistance of concrete of the same thickness.)

λ = factor in modified Berggren equation
= 0.97 (this is a conservative assumption).

I_f = thawing index at floor surface
= (60 - 32) (150) = 4,200 degree-days.

L = latent heat of gravel $= 144\ \lambda_d \frac{w}{100} =$
$144(125)(0.025) = 450$ Btu/ft³.

Then

$$X = (0.85)\ (12.5) \left(\sqrt{1 + \frac{(48)\ (0.97)^2\ (4{,}200)}{(0.85)\ (450)\ (12.5)^2}} - 1 \right) = 11.0\ \text{ft.}$$

Thus, the total amount of heat to be removed from the gravel pad by cold-air ventilation during the freezing season with ducts open is equal to the latent and sensible heat contained in the thawed pad. The heat content per square foot of pad is determined as follows:

Latent heat = (X) (L) = (11.0) (450) = 4,950 Btu/ft'.

Sensible heat (10% of latent heat, based upon experience)

= 495 Btu/ft2

Total heat content

= 5,445 Btu/ft'.

The ducts will be open during the freezing season (215 days), and the average rate of heat flow from the gravel during this season is equal to 5,445/(215 x 24) = 1.0 Btu/ft' hr. The average thawing index at the surface of the pad is

$$\frac{LX^2}{48\lambda^2 K} = \frac{(450)\ (11.0)^2}{(48)\ (0.97)^2\ (0.85)} = 1{,}420 \text{ degree-days}$$

This thawing index must be compensated by an equal freezing index at the duct outlet on the surface of the pad to assure freezeback. The average pad surface temperature at the outlet equals the ratio

$$\frac{\text{Required freezing index}}{\text{Length of freezing season}} = \frac{1{,}420}{215}$$

$$= 6.6°\text{F below } 32°\text{F or } 25.4°\text{F.}$$

The inlet air during the freezing season has an average temperature of

$$\frac{\text{Air-freezing index}}{\text{Length of freezing season}} = \frac{4{,}000}{215}$$

$$= 18.6°\text{F below } 32°\text{F or } 13.4°\text{F.}$$

Therefore, the average permissible temperature rise TR along the duct is (25.4 - 13.4) = 12.0 F.

The heat flowing from the floor surface to the duct air during the winter is equal to the temperature difference between the floor and duct air divided by the thermal resistance between them. The thermal resistance

$$R = \frac{X_c}{K_c} + \frac{X_i}{K_i} + \frac{1}{h_{rc}} = \frac{14}{(12)\ (1.0)} + \frac{4}{(12)\ (0.033)} + \frac{1}{1.0}$$

$$= 12.3\ \text{hr ft}^2\ \text{°F/Btu}$$

where

X_c = thickness of concrete, feet.
X_i = thickness of insulation, feet.
h_{rc} = surface transfer coefficient between duct wall and duct air.

where

(For practical design purposes hrc = 1.0 Btu/ft' hr °F, and represents the combined effect of convection and radiation. At much higher air velocities, this value will be slightly larger; however, using a value of 1.0 will lead to conservative designs.)

The average heat flow between the floor and inlet duct air is [(60 - 13.4)/12.3] = 3.8 Btu/ft: hr, and between the floor and outlet duct air is [(60 -25.4)/12.3] = 2.8 Btu/ft2 hr. Thus, the average rate of heat flow from the floor to the duct air is [(3.8 + 2.8)/2] = 3.3 Btu/ft2 V hr. As previously calculated the average heat flow from

*R_f = Reciprocal of time rate of heat flow through a unit area of a given temperature difference per unit thickness

the gravel pad to the duct air is 1.0 Btu/ft2 hr. The total heat flow X to the duct air from the floor and gravel pad is (3.3 + 1.0) = 4.3 Btu/ft' hr. The heat flow to the duct air must equal the heat removed by the duct air.

Heat added = heat removed

$\phi\, l m = 60 V A_d \rho C_p T_R$

Thus, the average duct air velocity required to extract this quantity (4.3 Btu/ft^2 hr) of heat is determined by the equation as shown below.

∅ = total heat flow to duct air, 4.3. Btu/ft2 hr
l = length of duct, 220 ft
m = duct spacing, 2.66 ft
A_d = cross sectional area of duct, 1.58 ft2
ρ = density of air, 0.083 lb/ft3 (fig. 4-30)
C_p = specific heat of air at constant pressure, 0.24 Btu/lb °F
T_R = temperature rise in duct air, 12°F.

Substitution of appropriate values gives a required air velocity.

$$V = \frac{\phi \ell m}{60 A_d \rho\ C_p\ T_R}\ \text{ft/min}$$

$$V = \frac{(4.3)\ (220)\ (2.66)}{(60)\ (1.58)\ (0.083)\ (0.24)\ (12.0)} = 111\ \text{ft/min.}$$

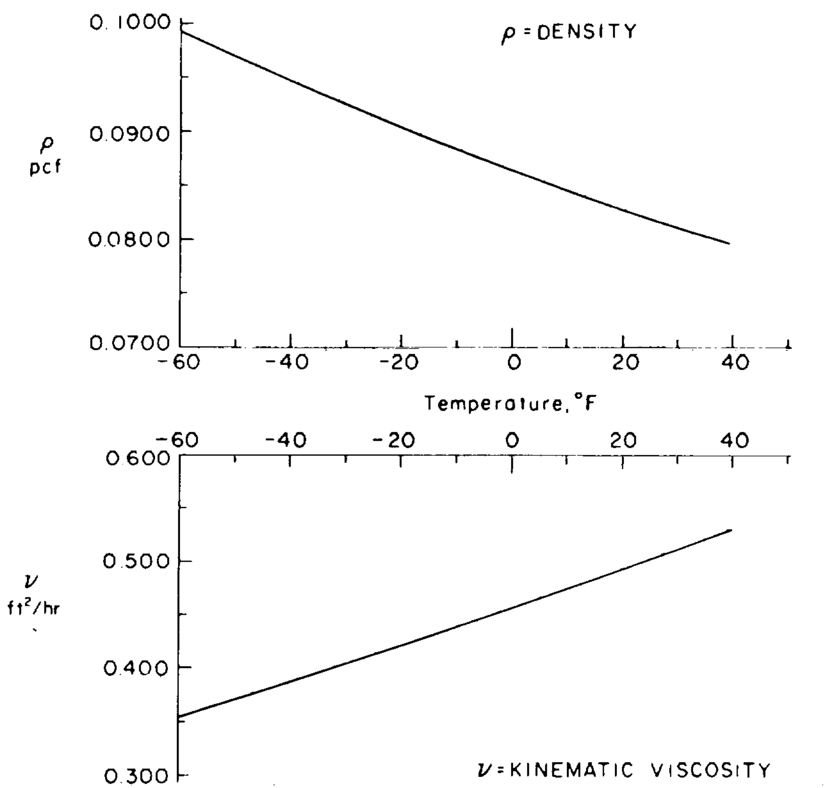

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Figure 4-30. Properties of dry air at atmospheric pressure[14].

The required airflow is obtained by a stack or chimney effect which is related to the stack height. The stack height is determined by the equation

$$h_d = h_v + h_f$$

where

$h_d = \frac{\rho \varepsilon H(T_c - T_o)}{5.2(T_c + 460)}$ in. of water natural draft head (ASHRAE Guide[117]).

ρ = density of air at average duct temperature, lb/ft^3

ε = efficiency of stack system, percent (this factor provides for friction losses within the chimney; ASHRAE Guide[117])

H = stack height, feet

T_o = temperature of air surrounding stack, °F

T_c = temperature of air in stack, °F

$h_v = \left(\frac{V}{4,000}\right)^2$ in. of water (velocity head)

V = velocity of duct air, ft/min

$h_f = f' \left(\frac{\ell_e}{D_e}\right) h_v$ in. of water (friction head)

f' = friction factor, dimensionless

ℓ_e = equivalent duct length, feet.

D_e = equivalent duct diamter, feet.

The technique used to calculate the friction head is as follows:

$$D_e = \frac{4 \text{ (cross-sectional area of duct, ft}^2\text{)}}{\text{perimeter of duct, ft}}$$

$$= \frac{4\ (1.58)}{\frac{2}{12}\ \frac{18 + 20}{2} + 12}$$

$$= 1.22 \text{ ft (ASHRAE Guide}^{117}\text{)}.$$

The equivalent length of the duct is equal to the actual length ℓ plus and allowance ℓ_b for bends and entry and exit. Each right-angle bend has the effect of adding approximately 65 diameters to the length of the duct, and entry and exit effect about 10 diameters (ASHRAE Guide[117]). The total allowance ℓ_b for these effects is [2(65 + 10)] = 150 diameters which is added to the length of the straight duct. The estimated length of straight duct ℓ_s is:

5 feet (assumed inlet open length)
220 feet (length of duct beneath floor)
15 feet (assumed stack height)
240

$$\ell_e = \ell_s + \ell_b$$

$$\ell_e = 240 + (150 \times 1.22) = 423 \text{ feet.}$$

The friction factor f' is a function of Reynolds number N_R and the ratio e/D_e.

A resonable absolute roughness factor e of the concrete duct surface is 0.001 ft, based on field observations. Suggested values of e for other types of surfaces are given in the ASHRAE Guide[117]. The effect of minor variations in e on the friction factor is small as noted by examining the equation below used to calculate the friction factor f'

Reynolds number is obtained from the equation

$$N_R = V \frac{a' + 0.25\ D_e}{\nu}$$

Where

$$N_R = \frac{(111 \times 60)\ (1.0 + 0.25 \times 1.22)}{0.49} = 17,700$$

V = average duct velocity, ft/hr

a' = shortest dimension, feet

ν = kinematic viscosity, ft^2/hr at 19.°F. (fig. 4-30).

The friction factor f' is obtained by solving the equation

$$f' = 0.0055 \left[1 + \left(20,000 \times \frac{e}{D_e} + \frac{10^6}{N_R}\right)^{1/3}\right] \text{ (ASHRAE Guide}^{117}\text{)}$$

$$= 0.0055 \left[1 + \left(20,000 \times \frac{0.001}{1.22} + \frac{10^6}{17,700}\right)^{1/3}\right] = 0.0285$$

Therefore the friction head

$$h_f = f' \times \frac{\ell_e}{D_e} \times h_v$$

$$= 0.0285 \times \frac{423}{1.22} \times h_v$$

$$= 9.8\ h_v$$

The draft head required to provide the desired velocity head and overcome the friction head is furnished by the chimney or stack effect.

The draft head h_d is obtained as follows:

$$h_d = h_v + h_f = h_v + 9.8\ h_v$$

$$= 10.8\ h_v$$

$$= 10.8 \left(\frac{V}{4,000}\right)^2$$

$$= 10.8 \left(\frac{111}{4,000}\right)^2$$

$$= 8.31 \times 10^{-3} \text{ in. of water}$$

The stack height required to produce this draft head is

$$H = \frac{5.2h_d(T_e + 460)}{\rho\varepsilon\ (T_c - T_o)}$$

$$= \frac{(5.2)\ (8.31 \times 10^{-3})\ (25.4 + 460)}{(0.083)\ \ (0.80)\ \ (25.4 - 13.4)}$$

$$= 26 \text{ ft.}$$

where

ρ = 0.083 lb/ft^3

T_c = 25.4°F

T_o = 13.4°F

ε = 80 percent (found to be reasonable design value based on observations over an entire season

h_d = 8.31 x 10^{-3} in. of water

If the stack height is too high for the structure, a greater thickness of foundation insulation could be used. In this example the effect of increasing the insulation thickness by one-half would result in lowering the stack height by five-eighths.

The first approximated stack height is next incorporated in the calculation of the length of straight duct l_s, and the newly obtained le is used to recalculate the friction head h_f. By iteration, the final calculated stack height is found to be 26.5 feet.

The stack height is an important variable as an increase in stack height will increase the duct airflow. Circulation of air through the ducts results from a density difference between the air inside the duct and that outside the building; a pressure reduction at the outlet end due to the stack effect; a positive pressure heat at the inlet end when wind blows directly into the intake stack opening; and a negative pressure head at the outlet when wind passes over the exhaust stack opening. If reliance is placed upon wind-induced draft for part of the required winter cooling, minimum wind conditions should be assumed in order to assure freezeback even in the least favorable winter. Vents should be cowled to take advantage of any available velocity head provided by the wind and as previously noted should be positioned to minimize snow infiltration. If sufficient air cannot be drawn through the ducts by natural draft, consideration may be given to such alternatives as placing, the exhaust stacks at the center of the building with intakes both sides, in order to reduce the effective duct length, or using a mechanical blower system to increase air circulation (however, see discussion of disadvantages of latter systems in (k) above. In order to minimize air flow resistances and to avoid differences in heat removal effectiveness between different parts of the foundation duct system, the number of ducts connected together through plenums into a single pair of intake and outlet stacks should not exceed three to five. Thus, an average foundation of this type may have numerous intake and outlet stacks. A chamber extending the length of the structure and open to the atmosphere along its length as shown in figure 4-24 may provide an acceptable alternative. Plenum chambers should be designed so as to permit ready access to the ends of the ducts for cleanout or other maintenance.

d. Foundation insulation. Foundation insulation may be used to control heat flow for the following objectives.

To control frost penetration and heave.
To reduce rates of thaw of permafrost and settlement.
For heating economy.
For comfort.
To control condensation.

(1) The general properties of insulating materials which are pertinent to construction under cold climatic conditions have been reviewed in paragraph 2-6d. Insulation used in foundations must satisfy the following performance criteria:

Provide required thermal insulating properties.
Provide adequate bearing capacity for static and dynamic loads which may be imposed.
Resist loss of thermal insulating properties and bearing capacity with time under the effects of moisture, ice and cyclic freeze thaw.

(2) Insulation can reduce quantity of heat flow but cannot prevent it entirely. Insulation should not be depended upon by itself to prevent thaw of permafrost under a continuously heated building or to prevent frost penetration under a continuously refrigerated warehouse or other structure.

(3) Where comfort is involved, as in quarters buildings, the insulation thickness should not be less than that required to maintain floor surface temperature at satisfactory comfort levels under design minimum winter temperature conditions. Floor temperature must also be maintained above the dew point of the interior air under these conditions; moist floors are not only unpleasant for personnel but may present hazards of slipperiness or sanitation. Much of the problem of cold floor discomfort in cold regions originates from cold air drainage from inadequately insulated ceilings, exterior walls, windows and doors. However, discomfort may be experienced even where these factors are absent, such as in interior rooms,

because the floor must necessarily be colder than the air above it whenever there is downward flow of heat through the floor, out of the building; otherwise heat flow would not occur. It is therefore most important for ventilated foundations of all types that adequate floor insulation be employed, together with a heating system that delivers enough heat near the floor to counteract so far as possible the effect of the heat transmission through the floor, as well as the effects of downdrafts from cold exterior surfaces. Systems which heat the floor itself can eliminate discomfort from heat loss through the floor but thus far have not been widely used because of cost.

(4) Some typical insulation designs which have been used in actual structures in Alaska and Greenland are shown in figures 4-15, 4-19 through 4-22 and 4-24 through 4-28. In general, the insulation amounts shown in these figures have provided only marginal comfort under winter conditions.

(5) When insulating materials are used in foundations, conditions may be extremely adverse for satisfactory performance of these materials. In insulation is installed below ground level under wet conditions, its value may be reduced or lost as a result of absorption of moisture (para 2-6d). If exposed to cyclic freeze-thaw, progressive physical breakdown of the insulating material may occur, again with increase of moisture content and with loss of thermal insulating and strength properties. In addition to use for controlling flow of heat from buildings into the foundations, insulating material may be used below ground level for a number of miscellaneous purposes such as to reduce the thickness of fill required to prevent freezing and thawing temperatures from penetrating into underlying frost-susceptible soil, to thermally protect buried pipelines or utilidors, or to control freeze and thaw penetration under and around bridge piers, grade beams, culverts or paved surfaces. In any such applications where detrimental moisture, ice, or freeze-thaw effects may be encountered, great care must be exercised in specifying type, placement and protection of insulation.

(6) Only closed-cell types of insulation should be used underground. Cellular glass may be used under high ground moisture conditions if it will be either continuously frozen or continuously thawed. Under these conditions no separate moisture barrier or protective membrane for the cellular glass is required. Cellular glass should not be used where it would be subject to cyclic freeze-thaw in presence of moisture. Where cyclic freeze-thaw in presence of moist (but not immersed) conditions is anticipated, it is recommended that only foam plastic closed-cell types of insulation, protected against absorption of moisture by a self-membrane or by sealed heavy polyethylene sheeting or equal, should be used. In soils of permanently very low moisture content, not subject to cyclic freeze-thaw, and protected against moisture infiltration and condensation or seepage by an overlying slab and/or other means, any closed-cell type insulation which has high integral resistance to moisture absorption may be used. Unless continuously frozen, installation of any type insulation where it will be below the water table should be avoided. Instead, alternatives should be sought, such as construction of the facility on a well-drained granular embankment where the insulation can be protected against moisture by the shelter of the structure itself and/or by embedment in high quality impervious concrete when appropriate.

(7) When insulation is used in a facility such as a hangar floor, where live loads occur, it must be placed at sufficient depth so that concentrated live load stresses will be reduced sufficiently to be within the allowable bearing values for the particular insulation used. At the same time the maximum allowable depth of insulation placement established by the live load stress in combination with the increase of overburden pressure with depth must not be exceeded.

(8) *Insulation of ventilated and ducted foundations.*

(a) For structures with open airspaces such as shown in figures 4-13 through 4-23 and ducted foundations such as shown in figures 4-24 through 4-28 insulation should be installed above the airspace or ducting system. This will not only minimize the amount of heat which must be removed by the foundation ventilation system but will provide excellent protection for the insulating material against moisture and freeze-thaw effects. Heat losses may be computed by the procedures of the ASHRAE Guide[117].

(b) For ducted foundations, trial computations for various alternative duct system and insulation design combinations will yield data on insulation requirements for foundation thermal stability. Computations of thicknesses required to maintain comfortable floor temperatures and to prevent condensation will provide additional input. From these data, a decision on insulation thickness can be made.

(c) For open airspace type ventilated foundations, comfort and heating economy will usually determine insulation thickness requirements.

(9) *Insulation of slab on grade and basemented foundations.*

(a) These types of construction are suitable for use in seasonal frost areas and thaw-stable permafrost without detrimental ground ice. Because of the susceptibility of insulating materials to moisture, ice and freeze-thaw effects, sufficient elevation above the natural terrain should be provided in such foundations, together with drainage, so that exposure of insulation to these adverse effects will be minimized. It should be noted that even where permafrost foundation materials are thaw-stable clean sands and gravels, the construction of a basement which will be kept at above freezing temperatures will tend to produce a sump in the frozen materials in which thaw water will tend to collect, causing a basement drainage problem.

(b) Two types of concrete floors are used in basementless houses: unheated floors relying for warmth on heat delivered above floor level, and heated floors containing heated pipes, ducts, or other integral heating system to constitute a radiant slab (panel heating). Thermal analysis should be made in accordance with recommendations contained in the latest edition of the ASHRAE Guide"'.

(c) Edge insulation for non-radiant concrete floor slabs on grade may serve the functions of preventing excessive heat losses at edges of floor slabs, maintaining comfortable floor temperatures for building occupants, and preventing condensation on floor surfaces adjacent to exterior walls.

(d) As shown in figure 4-31, the maximum rate of heat loss from an uninsulated non-radiant floor slab occurs at the edge, tending to result in an uncomfortably cold floor along the building perimeter. Condensation on the floor may result if the relative humidity is sufficiently high. Areas such as kitchens and mess halls where high relative humidity can normally be expected are more susceptible to condensation than areas such as warehouses and living quarters. A relatively small amount of insulation properly placed can reduce these adverse effects significantly.

(e) Edge insulation for slab-on-grade construction may be installed either horizontally as in figure 4-32 or vertically as in figure 4-33 with approximately equal results. The flow nets in figures 4-32 and 4-33 are theoretical, for assumed steady state heat flow conditions, seldom if ever fully realized in the field. The flow net in figure 4-33 is based on thermocouple measurements from an actual foundation, with some extrapolation. Since footings of slab-on-grade type buildings in frost areas are normally placed several feet deep in the ground in order to be below the seasonal frost line, the insulation is easily installed on the foun-

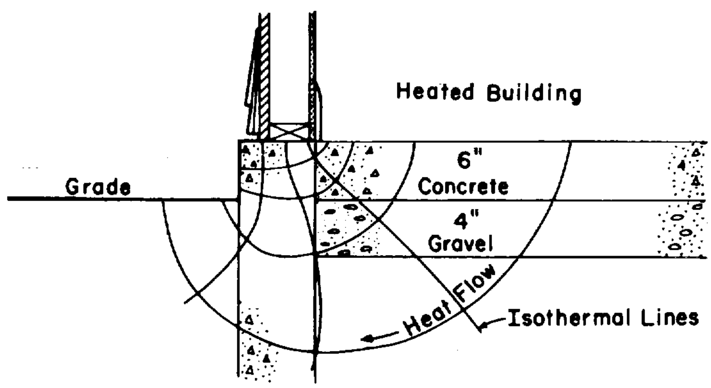

Figure 4-31. Flow net, concrete slab on grade, uninsulated[41].

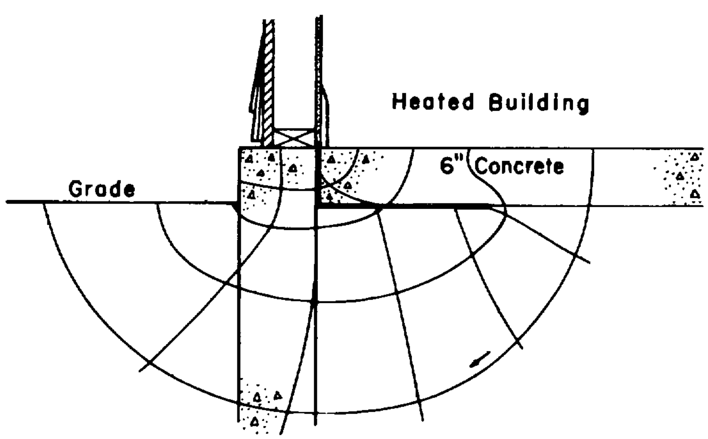

Figure 4-32. Flow net, concrete slab on grade, insulated[41].

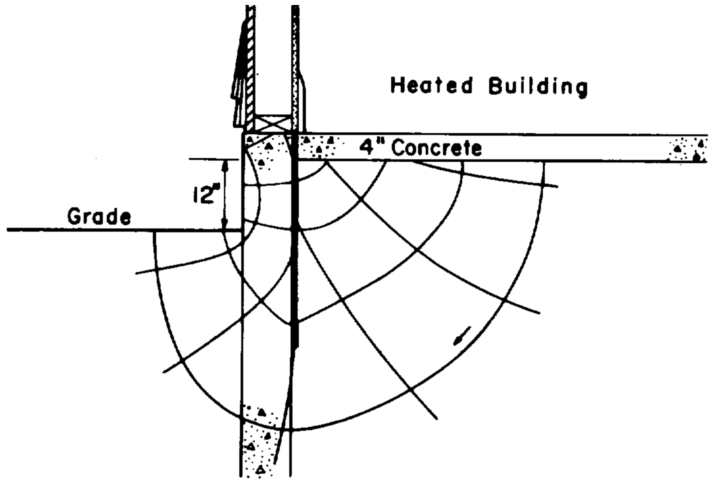

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Figure 4-33. Flow net, extrapolated from field measurements, with 1-1/2-inches cellular glass insulation, 36 inches long in vertical position[41].

dation wall. If placed horizontally under the slab, difficulty and expense are involved in leveling the base course sufficiently to provide uniform support under the insulation, particularly when the material contains gravel and cobble sizes. Because the compressive characteristics of the insulation are normally different than for the soil which underlies the slab in noninsulated portions of the foundation, non-uniform support tends to result, which is conducive to cracking of the slab, particularly if the slab is heavily loaded. This problem is avoided in the vertical type of insulation. Again the possibility of frost heave is reduced with the vertical insulation since flow net analysis (fig. 4-32 and 4-33) shows that the horizontal insulation gives greater opportunity for frost penetration under the floor slab. The vertical insulation can also be inserted as a single piece whereas the horizontal type requires a joint where the insulation meets the vertical piece placed between the wall and the slab. If both horizontal and vertical insulation should be used, the result would be a more complicated and expensive installation without corresponding gain in effectiveness. While placing vertical insulation on the exterior rather than the interior side of the slab-on-grade foundation wall would appear to offer some advantages, such as reduction of thermal stresses in the wall, it also presents the disadvantages for the insulation of more severe conditions of moisture, freeze-thaw and possibly frost heave. Also, in a design situation such as shown in figure 4-33, the insulation would have to be exposed above the grade level in order to attain continuity with the building insulation and this would be susceptible to damage. Therefore, placement on the interior face is recommended.

(f) From the condensation standpoint there seems to be normally no advantage in extending the insulation to the depth of maximum seasonal frost penetration since extending the insulation below the depth of balanced design(') has relatively little effect upon the floor slab temperatures . However, if there is a possibility of freezing of the soil under the slab adjacent to the inside face of the foundation wall as estimated by flow net analysis, insulation should be carried as necessary toward the maximum depth of freezing unless the soil and/or moisture conditions are such that no frost heave expansion can occur. Even slight heaving at the edge of the floor may result in cracking of the floor slab and may break any utilities passing through the slab. Usually, such depth-of-freezing insulation need only be of nominal thickness below the upper 2 or 3 feet. Measurements of actual frost penetration and temperatures under existing buildings in the area may be helpful when in doubt.

(g) Freezing may penetrate within the foundation wall of the slab-on-grade construction and cause difficulty if the building is not heated to normal temperatures or is not heated at all, if backfill of low insulating value, such as free draining gravel or crushed stone, is used on the outside of the foundation wall, or if insulation is insufficient for the conditions. For best resistance to frost penetration, the exterior backfill should consist of a relatively fine-grained and moist soil. A substantial snow cover on the ground adjacent to

([1])Balanced design is considered to exist where resistance to heat flow at the edge of the floor slab is the same whether the heat flows through the insulation or around the lower edge of the insulation.

the building will minimize frost penetration but usually cannot be relied on for design purposes. In some winters snow cover at time of maximum freezing conditions may be very small. Again snow will be absent if sidewalks are placed adjacent to the outside of the foundation wall and are kept shoveled or plowed and possibly if a large roof overhang is used. Sometimes wind patterns near the building may blow the area adjacent to the foundation wall essentially free of snow cover.

(h) Insulation should be specified in depths corresponding to commercially available dimensions. Avoidance of the necessity for field cutting is especially advantageous if factory-enclosed insulation board is available, since a minimum of on-the-job membrane resealing is then required. A record of actual measured floor temperatures 6 in. from the inside wall of the structure shown in figure 4-33 is presented in figure 4-34 [41]. Because the external temperature varied constantly during the period of observation, true steady state conditions were not achieved. Figure 4-35 shows floor temperatures vs distance from the interior wall determined by actual measurement at the same Loring AFB building, compared with results predicted by flow net analysis and electrical analog analysis and with adjusted comparative data from National Bureau of Standards studies. The actual measured values show more rapid drop in floor temperature as the exterior wall is approached than is predicted by the flow net and electric analog analyses. This difference is believed to be the

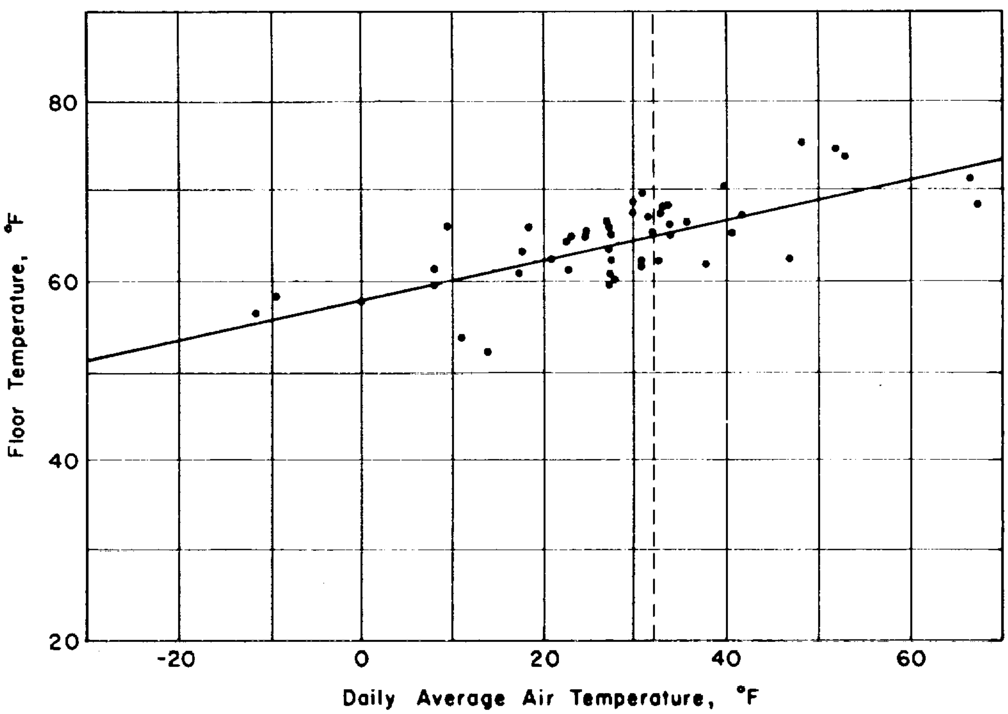

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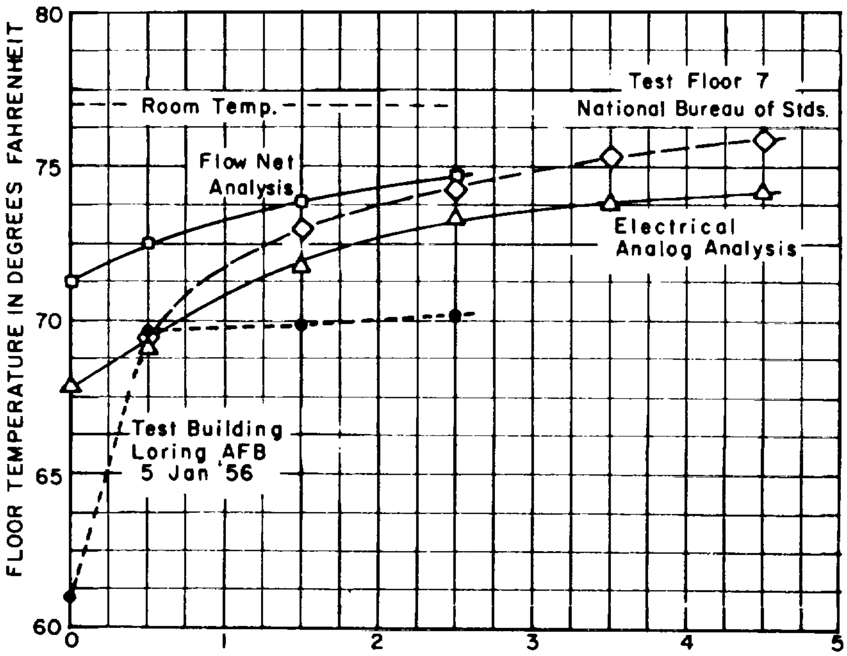
Figure 4-34. Floor temperature 6 in. from wall vs daily average air temperature measured at Loring AFB, Maine[41]. (Concrete slab 12 inches above grade; insulation is cellular glass 1 ½ inches thick, 36 inches vertical length.)



DISTANCE FROM WALL IN FEET

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Figure 4-35. Comparison of predicted and measured floor temperatures for a concrete slab on gravel with vertical insulation (same case as fig 4-33 [41]). Insulation 1 1/2-inches x 36 inches cellular glass except 3/4-inches x 18-inches rubber board for Test Floor 7. Room temperature 77°F. Outside temperature 32°F except 35°F in Loring AFB measurements. Data for test floor 7[204] adjusted from lower room temperature of 70°F by adding 7°F to all temperatures (from Government sources).

result partly of simplifications in the assumed boundary conditions for the flow net and electrical analog analyses, partly of such factors as localized cooling by the downward movement to the floor of cold air on the inside face of the exterior wall of the field test facility, and partly of the fact that the analytical approaches assume steady state conditions but these are probably never achieved in the field situation.

(i) If it is assumed that the air temperature in a mess hall or kitchen is 70 °F and the relative humidity is 70 percent, the floor surface temperature can not be lower than 60°F if the floor is to remain condensation free. For quarters or similar areas, a relative humidity of 40 percent and air temperature of 70°F will allow floor surface temperature to drop to 45 OF without condensation. Using these criteria, the curves of required insulation thermal resistance versus design winter 4-50 temperature in figures 4-36 and 4-37 were developed from theoretical studies of heat flow by an electrical analog method, verified in part by field data. Both figures show curves for no condensation over 6 in. from the edge of the wall. Recommended practice for insulation of concrete slab-on-grade structures is presented in table 4-3.

(j) Basements are desirable in areas of deep seasonal frost because heat losses tend to prevent or reduce frost grip on perimeter walls and foundations; however, a basement or underground facility in permafrost may be a source of structurally dangerous heat loss. Heat losses and wall and floor surface temperatures in partial or full basement or below-grade heated spaces may be calculated by procedures outlined in the ASHRAE Guide[117], extrapolated as necessary to arctic and subarctic temperature ranges, and insulation

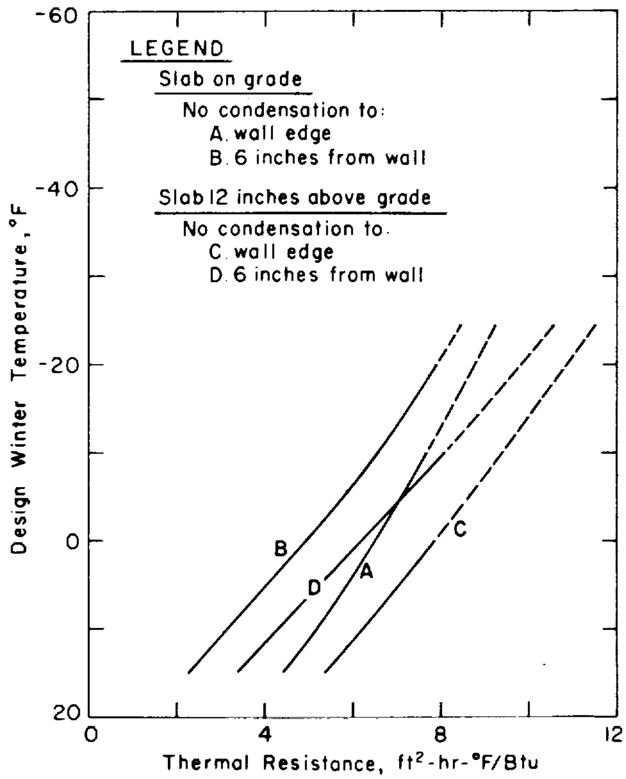

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Figure 4-36a. Thermal resistance vs design winter temperature for various vertical lengths of insulation for kitchens and mess halls (from electrical analog analyses)[41]. (length 24 inches)

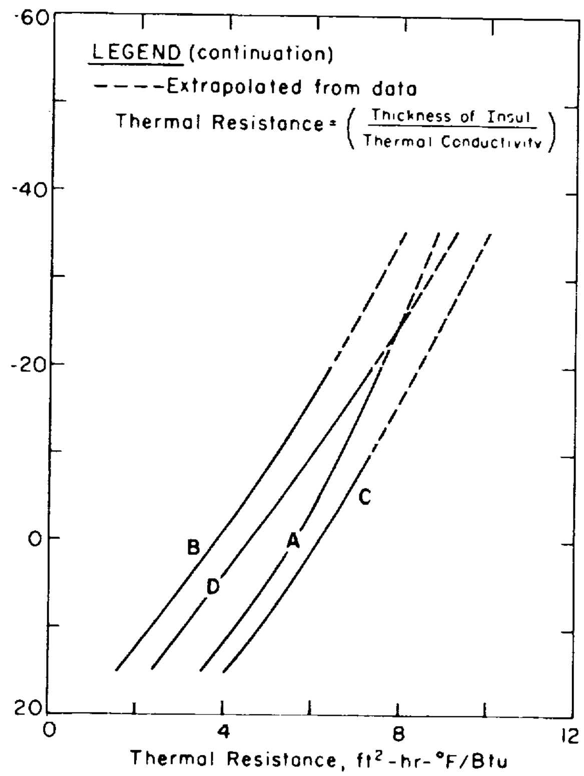

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Figure 4-36b. Thermal resistance vs design winter temperature for various vertical lengths of insulation for Kitchens and Mess Halls (from Electrical Analog Analyses)[41] (length 36 inches)

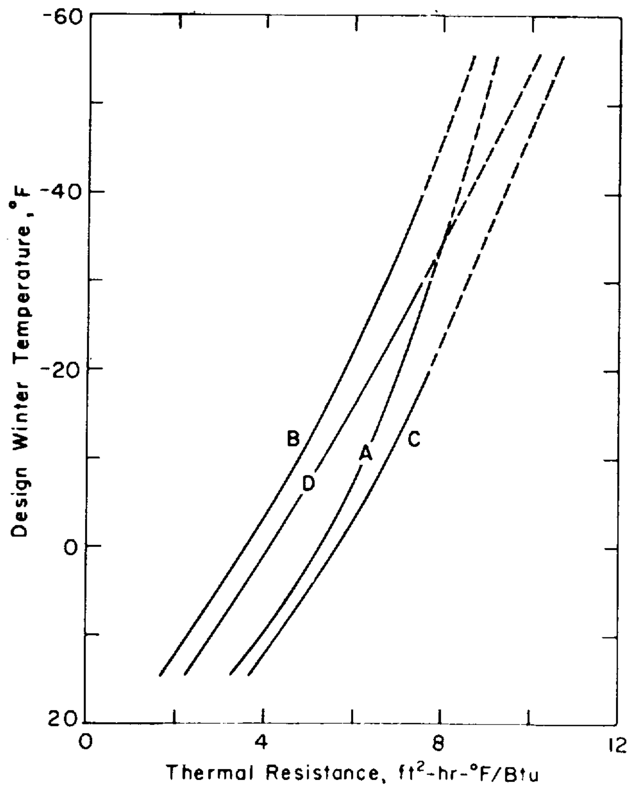

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Figure 4-36c. Thermal resistance vs design temperature for various vertical lengths of insulation for Kitchens and Mess Halls (from Electrical analog Analyses)[41] (length 48 inches)

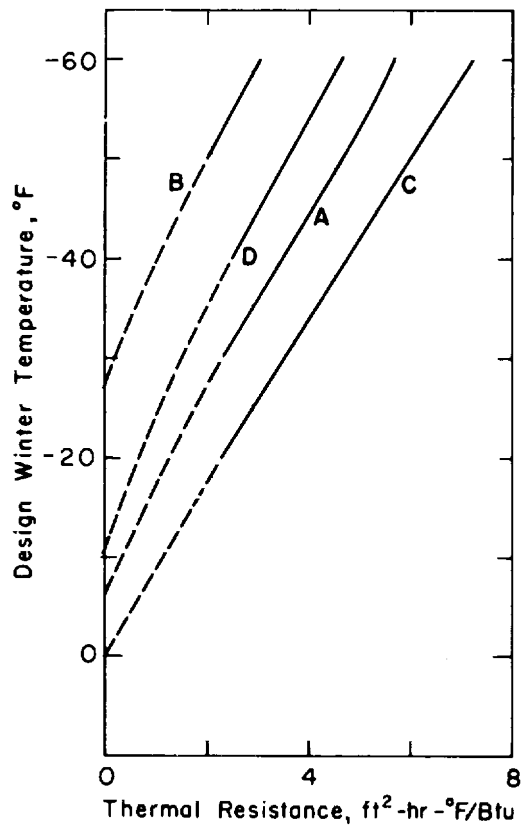

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Figure 4-37a. Thermal resistance vs design winter temperatures for two vertical lengths of insulation for barrack buildings (from electrical analog analyses)[41]. See figure 4-36 for legend. Note: TM 5-810-1[4], Mechanical Design-Heating, Ventilating and Air Conditioning, states: "1-03 OUTSIDE DESIGN TEMPERATURES. The outside heating design temperature should be determined in accordance with TM 5-785/AFM 88-29. Winter design for military installations will normally be based on the 97-1/2 percent dry-bulb temperatures tabulated in TM 5-785/AFM 88-29 and defined therein under 'General Information'. " TM 5-785/AFM 88-29, defines this temperature as the dry bulb temperature which is equalled or exceeded 97-1/2 percent of the time, on the average during the coldest three consecutive months. (Length 12 inches)

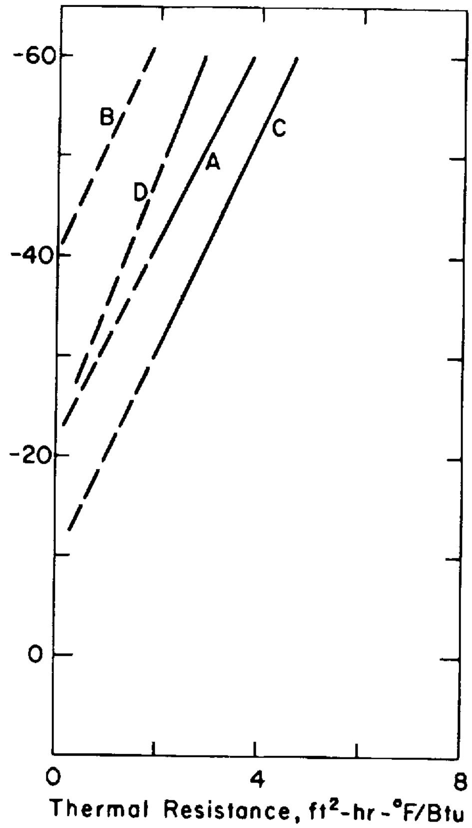

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Figure 4-37a. Thermal resistance vs. design winter temperatures for two vertical lengths of insulation for barrack buildings (from electrical analog analyses)4'. See figure 4-36 for legend. Note: TM 5-810-1, Mechanical Design-Heating, Ventilating and Air Conditioning, states: "1-03 OUTSIDE DESIGN TEMPERATURES. The outside heating design temperature should be determined in accordance with TM 5-785/AFM 88-29. Winter design for military installations will normally be based on the 9742 percent dry-bulb temperatures tabulated in TM 5-785/AFM 88-29 and defined therein under 'General Information'. " TM 5-785/AFM 88-29, defines this temperature as the dry bulb temperature which is equaled or exceeded 97% percent of the time, on the average during the coldest three consecutive months. (Length 24 inches).

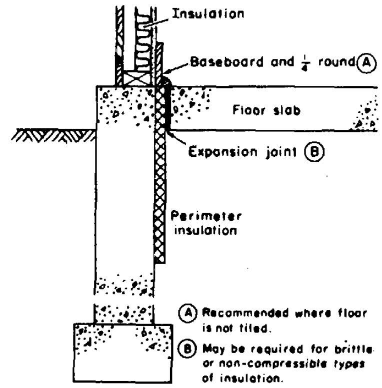
Table 4-3. Recommended Perimeter Insulation for Various Design Winter Temperatures

Structure Usage	Floor Surface Moisture to be Permitted	Elevation of Bottom of Floor Slab	Design Winter Temperature											
			0°F		-10°F		-20°F		-30°F		-40°F		-50°F	
			Vertical Length*	Thermal Resistance	Vertical Length	Thermal Resistance	Vertical Length	Thermal Resistance	Vertical Length	Thermal Resistance	Vertical Length	Thermal Resistance	Vertical Length	Thermal Resistance
Barracks, administrative buildings, etc. not subject to high humidity in winter. (Dew point temperature = 45°F.)	No condensation to wall edge	On grade	--	--	12"	1	12"	2	12"	3	24"	2	24"	3
		12" above grade	--	--	12"	1	12"	3	12"	4	24"	3	24"	4
	No condensation to 6" from wall	On grade	--	--	--	--	--	--	--	--	12"	1	12"	2
		12" above grade	--	--	--	--	12"	1	12"	2	12"	3	12"	4
Kitchens, mess halls, etc. subject to high humidity in winter (Dew point temperature = 60°F.)	No condensation to wall edge	On grade	24"	7	24"	8	36"	8	36"	9	48"	9	48"	9
		12" above grade	24"	8	24"	10	36"	9	36"	10	48"	10	48"	11
	No condensation to 6" from wall	On grade	24"	5	24"	7	36"	7	36"	8	48"	8	48"	9
		12" above grade	24"	7	24"	8	36"	8	36"	9	48"	9	48"	10

NOTES:

1. Only insulating materials that are not adversely affected by moisture and freezing or that are adequately protected against the moisture should be used.
2. $\text{Thermal resistance} = \dfrac{\text{Thickness of Insulation}}{\text{Thermal conductivity, K, of Insulation}}$, with units, $ft^2\text{-hr-}^\circ F/BTU$. For insulation with $K = 0.5\ BTU/ft^2\text{-hr-F/in.}$, one inch thickness has a Thermal Resistance $= 1/0.5 = 2\ ft^2\text{-hr-F/BTU}$.
3. "Design Winter Temperature" is defined as the temperature equaled or exceeded during 97½% of the hours in December, January and February.

* From top of floor slab.



requirements, if any, may be computed as necessary. It should be kept in mind that if heat escaping through the foundation walls of an uninsulated basement is sufficient, it may prevent soil freezing at the outer face of the wall but if insulation is then added on these walls soil adfreeze may occur, with possible risk of frost heave in frost-susceptible soils.

(k) If a basement is completely below grade and is not heated, the temperature in the basement normally will range between that in the rooms and that of the ground. The exact temperature which will naturally exist in an unheated basement or in crawl spaces below floors is indeterminate and depends on such things as the proportion of basement which is below ground, the number and size of windows or wall vents, the amount of warm piping present, the extent of piping or floor insulation, and the heat given off by a basement heating plant. If the floor in the space above is at all cold, the using service or resident will try to increase the floor temperature where it is not difficult to do so. It is necessary, therefore, for the design engineer to evaluate the probable conditions carefully and to make a realistic basement or crawl space design temperature assumption in accordance with his best judgment.

e. Granular mats.

(1) In areas of both deep seasonal frost and permafrost, a mat of non-frost-susceptible granular material placed at the start of the field construction effort on the areas of planned construction serves to moderate and control seasonal freeze and thaw effects in the foundation soils, to provide stable foundation support and to provide a working platform on which construction equipment and personnel may move and operate with minimum difficulty regardless of seasonal conditions. The mat becomes the locus of part or all of the seasonal freeze and thaw action, reducing or eliminating these effects in the underlying in-place materials. Its thermal function is more nearly that of a heat sink than of an insulator, dampening the effects of seasonal fluctuations relative to the subgrade. The mat reduces the magnitude of any seasonal frost heave in underlying materials through its surcharge effect. It is usually convenient to place the mat at the start of the construction so as to serve both working platform and structure foundation purposes.

(2) To insure a dry, stable working surface during upward flow of melted water in the thaw period, the mat materials must be sufficiently pervious to bleed water away laterally without its emerging on the surface (TM 5-820-2/AFM 88-5, Chap. 2 [8]). The mat is most commonly composed of clean, well-graded bank run gravel of 2 and 3 inches maximum size, offering good compaction, trafficability and drainage characteristics. Where such material is not available, alternatives such as crushed rock or clean sand with a soil-cement surface will have to be considered. Materials should contain sufficient sand sizes to retain some moisture; this will help to control thaw and freeze penetration. If a very coarse gravelly mat material is to be placed over a fine-grained subgrade, a sub-base of 6 in. minimum thickness of dean, non-frost-susceptible sand should be placed directly on the subgrade in order to minimize the possibility of upward intrusion of fines into the mat during thaw periods. Increased volumetric latent heat of fusion corresponding to the higher moisture-holding capacity of this sand will also help reduce freeze and thaw penetration into the subgrade.

(3) When the mat is needed only as a construction working platform, and control of freeze and thaw penetration into the subgrade is not a factor, the mat is made only thick enough to carry the loadings which may be applied to it during actual facility construction, during critical periods of reduced subgrade strength. For this purpose, thicknesses should be determined from the flexible pavement design curves given in TM 5-818-2[6] or TM 5-852-3[12]. However, under summer soft ground conditions as much as 3 ft of material may need to be placed by end dumping and spread in one layer simply to support the hauling equipment initially. If compressible materials underlie, even more may be needed to meet design grades. Under other conditions, as where the natural soils are free draining sands and gravels, little or no mat may be needed except to provide, through elevation, a well-drained surface during unfavorable periods of the year, to provide a uniform work platform level, or to minimize snow accumulation problems.

(4) Figures 4-16, 4-17, 4-21 and 4-23 illustrate use of mats to provide thermal protection to the foundation. Figures 4-13, 4-14, 4-15, 4-22 and 4-24 through 4-27 illustrate use of mats additionally to provide stable bearing support for footings and rafts. Figure 4-28 shows both thermal protection of a pile foundation supported in the natural ground and support of the hangar pavement on the gravel mat. When used for foundation support the mat should be designed with sufficient thickness to achieve any needed heave reduction through its surcharge effect. For temporary facilities or for light flexible structures, complete frost heave control may not be necessary or economical. For facilities which require a foundation free from any frost heave or thaw settlement effects the mat should be made thick enough so that seasonal freeze and thaw are kept within the mat if the subgrade conditions are unfavorable. Required thicknesses for thermal control should be computed by procedures outlined in TM 5-852-6/AFM 88-19, Chapter 6 [14]. Approximate values may be estimated from figure 4-4. If thicknesses are excessive, alternative foundation design approaches must be investigated. It should be kept in mind that during the thawing season building heat may add to the thawing in-

dex of air passing through a ventilation space so long as the air temperature is below the building temperature even though the temperature differential is smaller and the thawing index increase is of smaller magnitude than the freezing index decrease in winter.

(5) The surface of the mat should extend outside the perimeter of the structure or any foundation members at least a distance of 5 feet before sloping down to the ground surface. In addition, greater width should be added for walkways or vehicular traffic or to handle construction equipment where required.

(6) Allowable bearing values for footings and sills supported on granular mats vary from 2,000 psf on shallow, poorly graded sandy mat materials to 6,000 psf on deeper, well-graded gravels and clean crushed rock. Thickness of granular material between footing and underlying natural soil must be sufficient to reduce concentrated stresses on the natural soils to tolerable levels, as discussed in paragraph 4-4.

f. Protection against solar radiation thermal effects.

(1) Solar radiation is a major factor in the thawing of frozen ground, particularly at very high latitudes. A very substantial proportion of the heat received in summer in those regions comes from this source.

(2) Because the portion of a granular foundation mat extending beyond the building perimeter tends to absorb substantial solar heat, thaw of permafrost under this extension, unless full protection against combined heat input is provided, may cause settlement of this portion of the mat in time. At the extreme perimeter of the mat where it is tapered down to the natural ground surface, the plane of seasonal thaw penetration tends to be depressed because of the thinned granular cover. On south-and west-facing embankment edges the effect is intensified by increased solar heat absorption. Settlement or even sloughing of the embankment edge may result, and ponding of water on thaw-depressed natural ground at the toe of the slope may further intensify the condition by increasing the absorption of solar radiation. Possible adverse effects on the construction and possible increased maintenance requirements from these perimeter conditions should be anticipated in the design. Typical provisions include extension of the mat sufficiently far beyond the structure foundations so that the latter cannot be affected by thaw settlements, including those from surface and subsurface drainage of thaw water; re-forming of the embankment cross-section with additional material from time to time during the life of the facility so as to gradually build up extra thickness of granular material at thaw-settlement locations; and use of heat-absorptive or heat-reflective coverings. The area of mat exposed to solar heat input is sometimes covered with moss or peat to minimize these effects, but because of the fire hazard contributed by the dry organic material in summer, it is policy to avoid this in Army construction except for the outermost edge of the mat contiguous with the natural terrain, where the organic material can reasonably be expected to support a live vegetative cover. Another solution is to provide a more reflective surface by using a light-colored reflective aggregate cover, if available, by spraying with whitewash or a very light spray coating of white paint, or by constructing white painted pavement in the proper position near the structure. Such methods will only be successful after construction activities have ended and accumulation of dirt and dust on the surface has ceased. An insulating course in the ground may sometimes be used to slow rate of thaw. Positive but potentially expensive solutions which shade both the granular pad close to the structure and the piles at the perimeter are special hinged plywood panels which can be extended from the side of the structure as shown in figure 4-18, or timber vanes as shown in figure 4-19; while acting as sunshades, these do not interfere with the flow of ventilating air under the structure. Actually, simple white painted moisture-durable panels of metal or wood or other material can be used, even if only laid on the ground surface in the critical locations and anchored down. Vegetation also can be an effective agent for control of radiation effects but may not be feasible within the time frame available or under the site climatic conditions. Experience indicates that protection is usually needed only on the south and west sides of structures, although facilities located well north of the Arctic Circle may in fact receive substantial sunlight on all sides during the height of summer. Areas of a well ventilated foundation which are fully shaded by the structure against sunlight may experience a rise of the permafrost table following construction. Cantilevering the edge of the structure beyond piling, post or column supports so as to provide positive shade for the foundation will help to assure maximum foundation stability on permafrost.

(3) Individual sun shades are sometimes used on the upper column part of an isolated steel pile. These consist of sheet metal enclosures, usually of aluminum alloy or reflective-painted light gauge steel. Care must be taken in fastening them to the column as wind damage has sometimes occurred. The air-space between enclosure and column plus the reflective surface of the enclosure is very efficient.

(4) Another expedient is to increase the pile length by about 2 feet to ensure adequate bond length in permafrost when summer heat may increase the thaw depth by up to 12 to 18 inches around a steel pile.

(5) Probably the simplest, yet adequate, approach for controlling direct radiation absorption by piling which may be exposed to the sun is to paint the exposed portions with a highly reflective white paint.

4-3. Control of movement and distortion from freeze and thaw. The foundation design engineer must establish the amounts of movement and distortion which may be tolerated in the structure supported by the foundation and must develop his design to meet these criteria. Foundation design must interrelate with design of the structure.

a. Movement and distortion may arise from seasonal upward and downward displacements, from progressive settlement arising from degradation of permafrost, from horizontal seasonal shrinkage and expansion caused by temperature changes, and from creep, flow, or sliding of material on slopes. Detrimental effects may consist of the following:

Tilting of floors
Jamming of doors and windows
Cracks and separations in floors, walls and ceilings
Breaking of glass Complete disorientation of structural stresses
Structural failures
Shearing of utilities within structure, at structure perimeter, and in the ground
Damage to installed equipment or interference with its operation

b. Small flexible buildings on posts and pads can easily tolerate several inches of seasonal movement provided the differential between various parts of the foundation is not excessive and provided utility connections have the requisite flexibility. Post and pad, footing and column, and simple pile foundations such as illustrated in figures 4-13 through 4-20 are generally satisfactory for such relatively light structures. However, permanent structures should be designed to be free from both seasonal and progressive movements or distortions during the life of the facility, except for the normal cyclic effects caused by seasonal expansion and contraction of ground and structural elements. For certain technically complex facilities such as radar or communications installations which can tolerate only minute movement, exceptionally stringent foundation stability and response requirements may be established.

c. As foundation loadings increase, support members must be more closely spaced and/or have heavier cross-sections, and footings must have larger areas, with less and less space between footings. Figures 4-21 through 4-23 show designs of higher load capacity. Where these systems may become uneconomical, designs such as the systems shown in figures 4-24 through 4-28, capable of very high unit loadings, should be considered. As a last resort, where even such air ducts cannot be incorporated into the foundation, tubing systems through which liquid refrigerant is circulated may need to be used in the foundation. The magnitude of the foundation loading will have an influence on the type of material used, and will determine the size and arrangement of foundation components. The relative in-place costs of materials such as wood, steel and concrete must be known for accurate economic evaluation of optional approaches. The geometrics, size and loading of the members of the foundation will determine the upward heave force and displacement pattern and adfreeze stresses developed. The foundation, structure, and loading will react, in turn, to restrict the amounts of actual heave displacement which can occur.

d. Wood frame structures have relatively high flexibility and capability for adjustment to differential foundation movements. They have often continued to give acceptable service even under conditions of such severe distortion that doors have required substantial trimming in order to open and close, window glass has required replacement with plastic film to reduce breakage, and floors have become conspicuously unlevel. Steel frame structures usually are highly suitable for modern military facilities, are highly reliable and versatile, and offer excellent long term service capability. They are more rigid than wood frame structures but more tolerant of movement than reinforced concrete and masonry. Both pad supported and fixed types of foundations may be used for wood and steel structures. Concrete and masonry exhibit in the cold regions about the same responses to distortion as in temperate or tropic areas. Foundations which provide complete freedom from cracking or distortion of the structure are required for concrete or masonry structures.

e. If the foundation materials are thaw-stable, clean, non-frost-susceptible granular soil or ice-free sound bedrock, the same type of foundations may be considered as would be employed in the temperate zones.

f. Figure 4-38 shows a typical record of permafrost degradation and vertical movements of a small wooden building constructed with a "floating" foundation consisting of a rigid concrete raft on a gravel pad. This particular design is not an economical one; it was constructed as an experiment",'". Although regular seasonal vertical movements occurred and there was gradual progressive degradation of permafrost at the warmer southwesterly side of the foundation which received maximum sunlight in the summer, resulting in tilting, the building itself performed excellently. warmer southwesterly side of the foundation which received maximum sunlight in the summer, resulting in tilting, the building itself performed excellently.

g. Figure 4-39 shows displacements for a small wooden building supported on wood piles embedded in permafrost",'". In this case, 1 of the 20 supporting piles failed to achieve freeze-back in permafrost and continued to heave progressively upward. Distortion of the wooden building of more than 6 inches occurred before the top of the pile was sawed off and a jack in

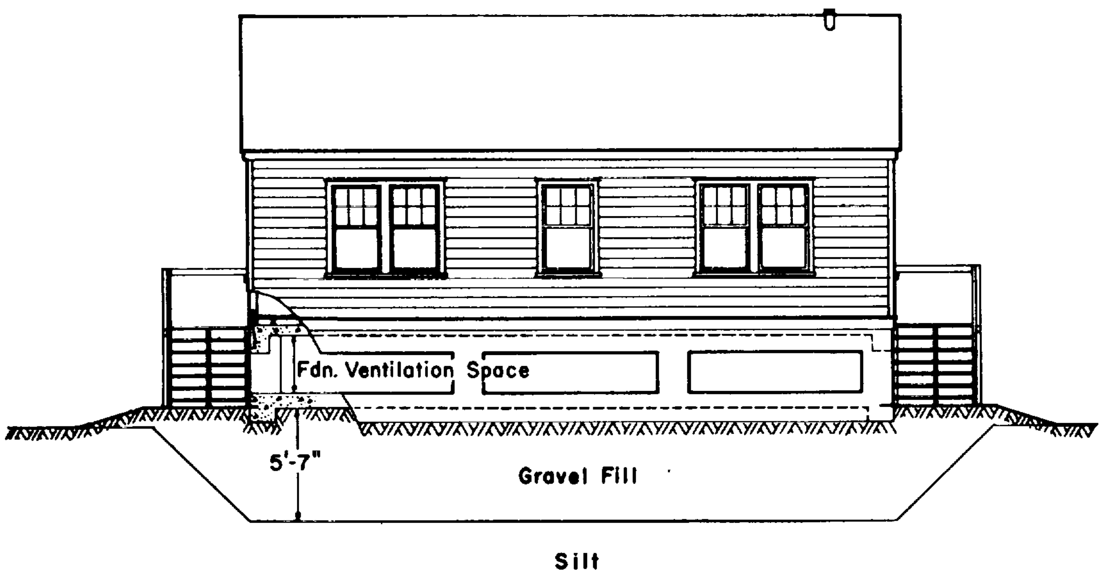


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Figure 4-38a. Wood frame residence 32 x 32 feet on rigid concrete raft foundation, Fairbanks, Alaska (South side Elevation)

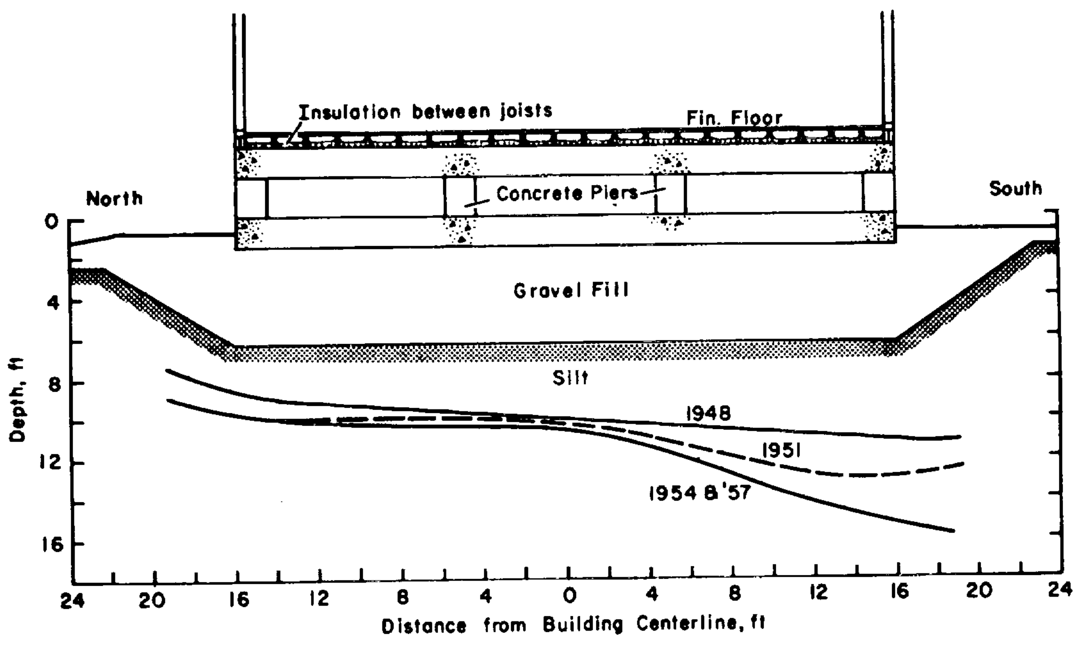

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Figure 4-38b. Wood frame residence 32 x 32 feet on rigid concrete raft foundation, Fairbanks, Alaska (Degradation of permafrost on N-S centerline, 1948-1957)

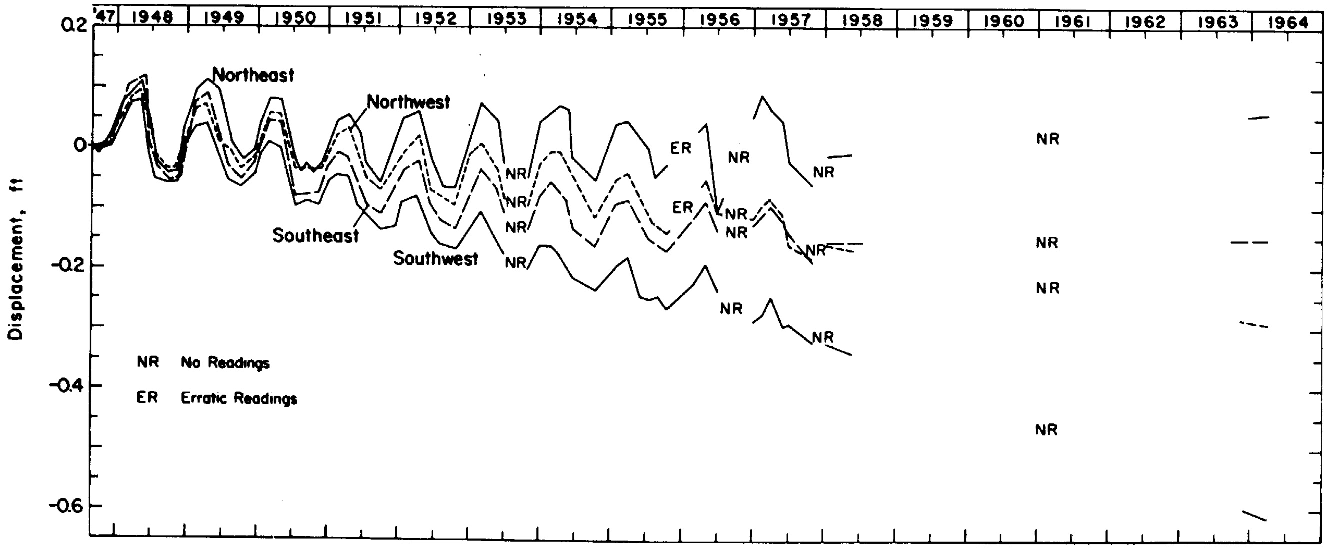

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Figure 4-38c. Wood frame residence 32 x 32 feet on rigid concrete raft foundation, Fairbanks, Alaska (Displacement of corners, 1947-1964) serted. The building is still giving good service more than 20 years after erection although the one jack requires periodic adjustment and occasionally an additional section has to be removed from the top of the pile. In figure 4-39 the rapid jacking out of the ground of porch posts embedded only in the annual frost zone is contrasted with the performance of the permafrost embedded piles supporting the building itself. These porches became unusable and had to be reconstructed.

h. Figure 4-40 shows the rapid settlement of a wooden garage building on a grossly inadequately ventilated foundation', "0. The greater displacement of the west side as compared to the east may have resulted from the effect of the late afternoon sun on the west side. Degradation of the permafrost was finally nearly arrested by discontinuing heating of the building. In spite of extreme differential distortion of the building it was then used for some years for storage with very little further distress. It was finally moved to a new foundation and reconditioned; it is still in use more than 20 years after its original construction.

i. Power transmission line towers can often tolerate several inches of upward and downward seasonal movement (depending on the tower design) provided there is no progressive tilting, heave or settlement. Therefore, it is often possible for such towers to be economically sup ported on footings resting on pads of free-draining granular materials. Radar or communication antenna towers, however, must usually maintain fixed orientations at all times and normally require fixed types of foundations. Bridge piers and abutments are normally thought of as fixed types of structures but recent observations of such construction in Alaska have shown that considerable seasonal movements may occur[48]. Figure 4-41 shows observed movements of a bridge pier in central Alaska. Nevertheless, such facilities must be designed for permanent stability, accepting small cyclic seasonal deflections due to thermal stresses. Water and POL storage tanks may accept considerable movement and distortion if properly constructed and may therefore be supported either on "floating" or fixed types of foundations, provided long range thermal stability is assumed.

j. When components such as exterior loading platforms, porches or unheated wings are attached to heated structures which are stable supported, the potential exists for severe differential movements between the facility components (fig 4-39). Such conditions must be carefully studied and precautions must be taken in design to ensure that structural damage will not occur. To achieve this, the foundation of the attached or unheated facilities should be provided with the same

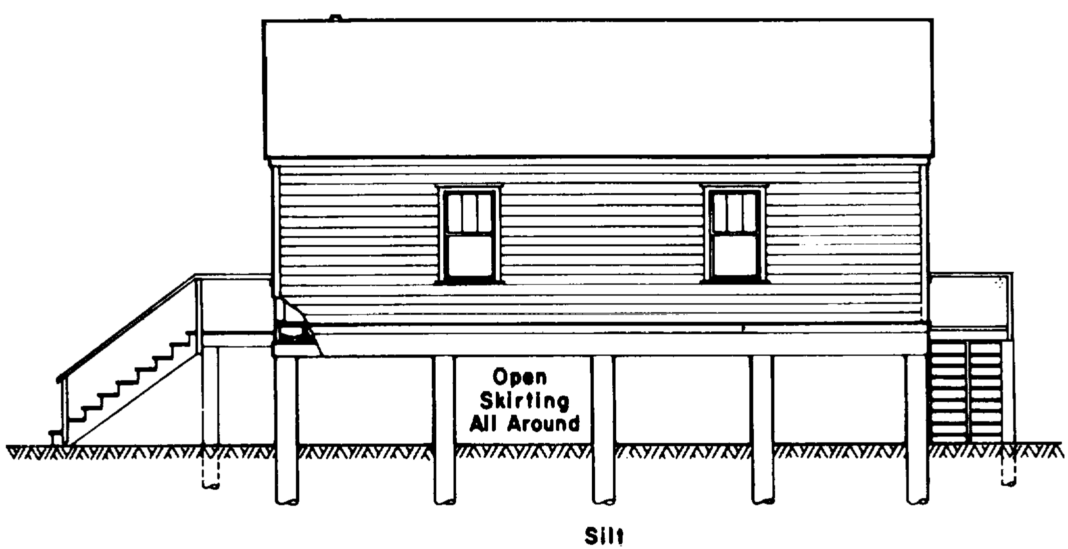

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Figure 4-39a. Wood frame residence 32 x 32 feel on wood pile foundation, Fairbanks, Alaska[73,180]. See figure 4-25 for site conditions. See figure 4-12b for degradation of permafrost, 1948-1957. (North side elevation.)

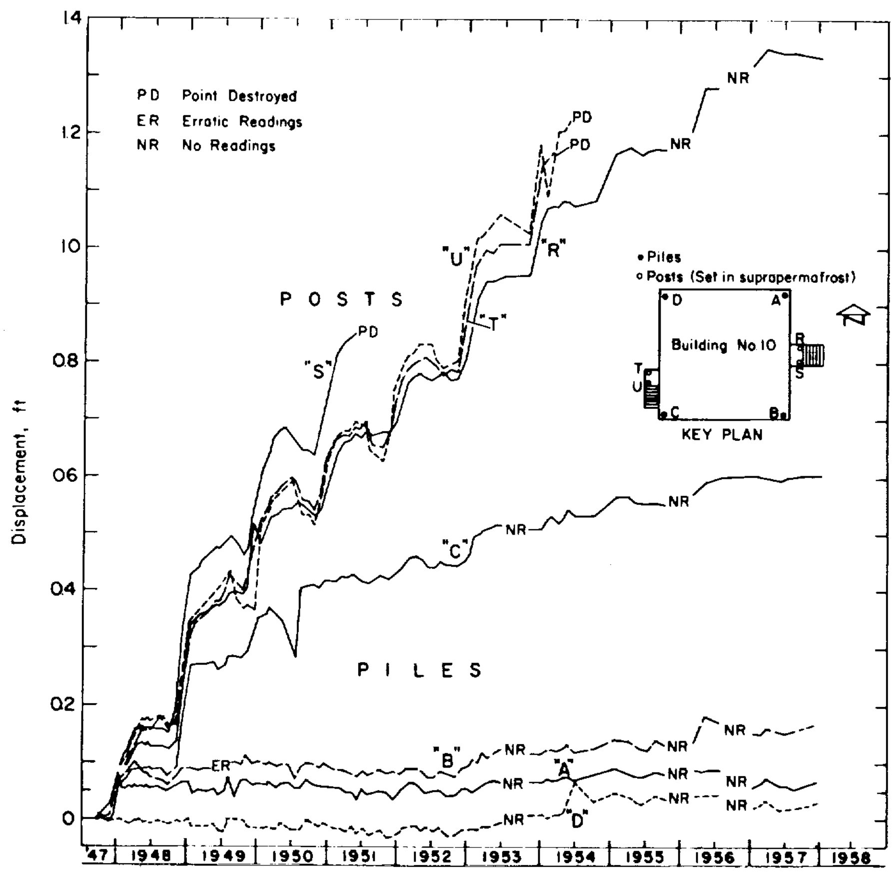

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Figure 4-39b. Wood frame residence 32 x 32 feet on woodpile foundation, Fairbanks, Alaska (Displacement of piles and posts, 1947-1958.)

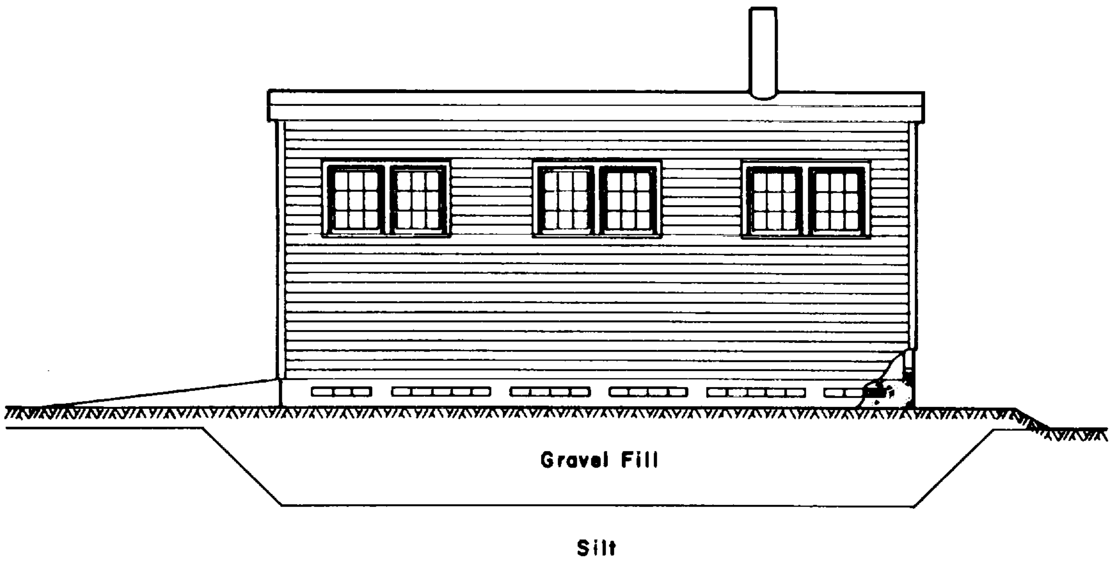

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Figure 4-40a. Wood frame garage, 32 x 32 feet on rigid concrete raft foundation, Fairbanks, Alaska[73,188]. See figure 4-25 for site conditions. (East side elevation.)

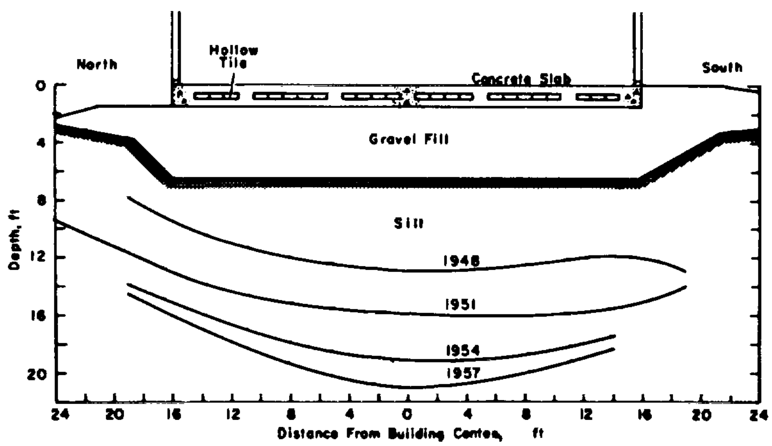

Figure 4-40b. Wood frame garage 32x 32 feet on rigid concrete raft foundation Fairbanks, Alaska (Degradation of permafrost on N-S centerline, 1948-1957)

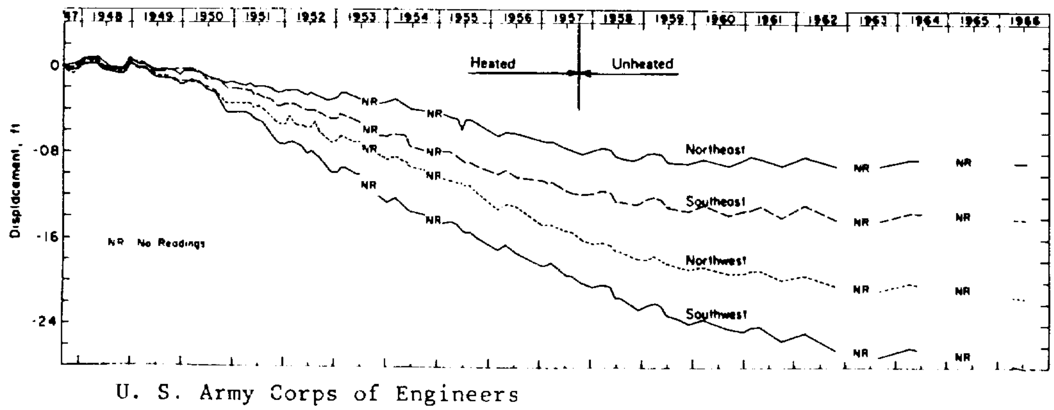

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Figure 4-40c. Wood frame garage, 32x 32 feet on rigid concrete raft foundation, Fairbanks, Alaska (Displacement of corners, 1947-1966)

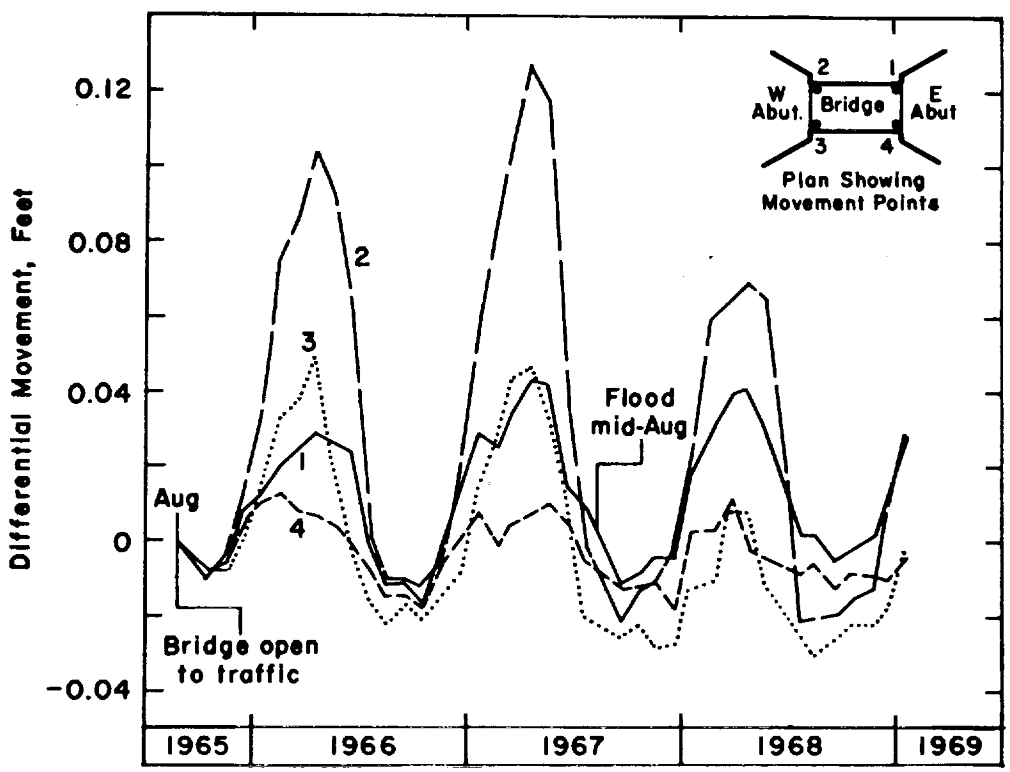

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Figure 4-41. Vertical movement of 50-feet single span bridge on pile-supported concrete abutments, east-west road near Fairbanks, Alaska[48]. (Initial foundation conditions: each abutment supported on four 10BP57 H-piles driven 28 feet into 4 to 6 feet unfrozen gravel fill, 15 feet frozen silt and 7 to 9 feet frozen gravel; permafrost degrading slowly since construction.)

degree of stability, with respect to frost heave or thaw settlement, as the main or heated parts of the structures.

k. Control of settlements which may result from thaw. Thaw penetration beneath a slab or foundation readily occurs non-uniformly. The resulting differential stresses and strains may cause distortion and cracking in the slab, foundation and structure. If thaw is rapid and imposed loadings are high, displacement of over-stressed, thaw-weakened foundation soil is possible. Under dynamic loading, pumping and mud boils are possible. Thaw water may emerge upward. These thaw settlement problems should be avoided by adopting the proper foundation design approach for the conditions and by designing for full thermal stability control, using the principles presented in paragraphs 4-1 and 4-2. If damaging thaw settlements should start to occur, a mechanical refrigeration system may have to be installed in the foundation or a continual program of jacking and shimming will have to be adopted together with installation of flexible utility connections.

l. Control of frost heave and frost thrust. When estimation of the depth of seasonal frost penetration and evaluation of frost uplift or thrust forces indicate potential problems, they must be taken fully into account in design and measures taken as necessary to avoid detrimental effects from either the structural stresses developed by the frost forces or from the resulting deformations.

(1) In the case of slab or footing foundations the heaving thrust of underlying frost-susceptible soils may act directly upward against the base of the foundation as illustrated in figure 4-42a or laterally against foundation walls as in figure 4-42b. Piers, posts, piles, or entire foundations surrounded by frost-susceptible soils are also subject to frost heaving as the ground adjacent to lateral surfaces is displaced annually as shown in figure 4-42c. Should the soil conditions, moisture availability, loading or frost penetration vary under the foundation the frost heaving effects will be non-uniform. to illustrate the problems involved, figure 4-43 shows the heave of floor slabs in wing hangars at Loring AFB, Maine". Through a combination of sufficient clearance of the slab around the interior columns and sufficient anchorage of the column footings, no movement of the

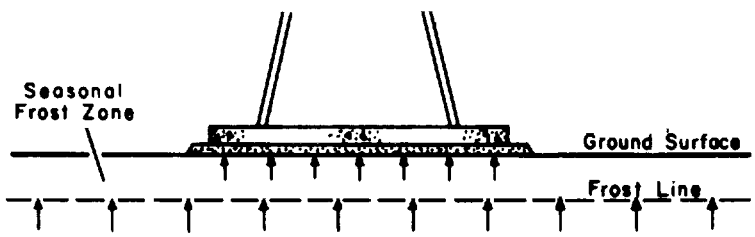

Figure 4-42a. Frost Action Effects (Heaving of soil in seasonal frost zone causing direct upward thrust on overlying structural elements.)

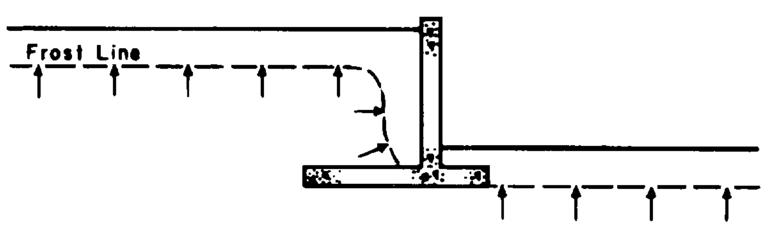

Figure 4-42b. Frost Action Effects (Freezing of frost-susceptible soil behind walls causing thrust perpendicular to freezing front.)

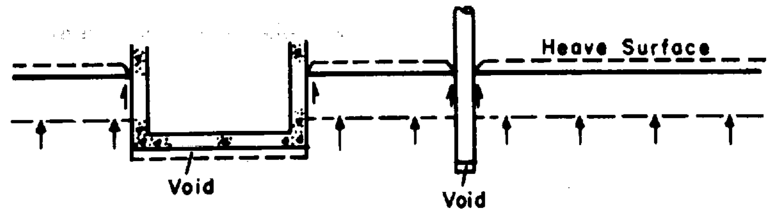

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Figure 4-42c. Frost Action Effects (Force at base of freezing interface tends to lift entire frozen slab, applying jacking forces to lateral surfaces of embedded structures, creating voids underneath. Structures may not return to original positions on thawing.)

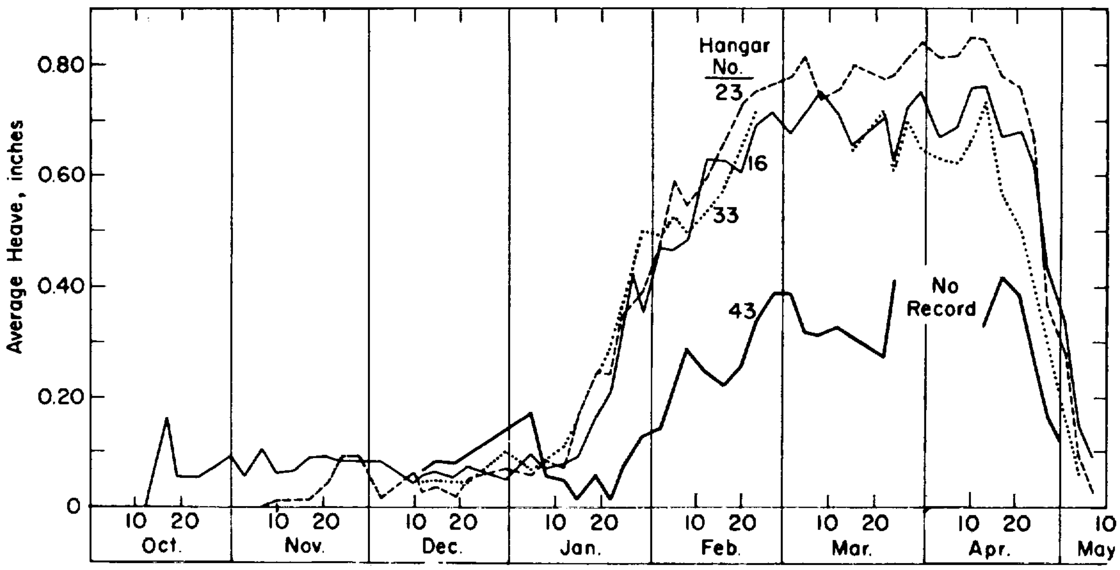

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Figure 4-43. Average heave vs. time. Floors of unheated wing hangars, Loring AFB, Limestone, Maine[38].

columns resulted and the slabs returned to their original grades in the spring. Figures 4-44 and 4-45 show the magnitude of heave forces actually developed on plain steel pipe and creosoted wood piles at Fairbanks, Alaska[51].

(2) By a nominal estimate of the effective area of the slab of seasonally frozen soil which contributes to heave or thrust forces on a structure and by use of the maximum heave pressure data presented in figure 2-9, and discussed in paragraph 2-4, a rough approximation of the total heave or thrust forces on a given structure can be made. A comparison of these forces with structural foundation loadings or passive resistive forces will give an indication of their relative balance. By laboratory tests[40,66], using either undisturbed or remolded materials as applicable for the particular frostsusceptible soil involved and by more rigorous analysis of the interaction of the structure and frost forces, a more accurate estimate is possible in theory. However, because of the many variables of soil conditions, moisture availability and frost penetration, precise quantitative predictions are not usually practical in the present state-of-the-art.

(3) When frost penetrates downward along vertical faces of walls, footings and piles in contact with earth, as illustrated in figure 4-42c, an adfreeze bond develops between the soil and the concrete, wood or other material of the foundation. If the soil is frost susceptible and heaves, the wall or footing tends to be lifted with the layer of frozen soil because of this adhesion. The weight of the structure at the same time tends to restrict frost heaving, the reduction diminishing with distance from the face. The maximum upward force which can be exerted on the structure is usually not limited by the uplift force which can be developed at the plane of freezing, but by the unit tangential adfreeze bond strength and the area of adfreeze contact on the wall, footing or pile itself. The total uplift force which is thus imparted to the structure is a function of the thickness of the frozen layer; the total force increases as the depth of frost penetration and the total bond contact area increase.

(4) The tangential shear stress which can be developed under a given rate of loading, or allowed in design, on the surface of adfreeze is limited by creep, which occurs down to stresses as low as 5 to 10 percent of the rupture strength measured under relatively rapid loading. Tangential shear stress is a function of such factors as the temperature, surface material (as concrete, steel, wood or paint), presence of salt or other chemicals in the soil moisture, direction and sequence of freezing, and rate and duration of loading. Tangential shear stress values are discussed in paragraph 4-8. In the present state of knowledge, dean metal, untreated smooth wood or smooth concrete surfaces may all be assumed to have similar adfreeze bond potentials. Rough concrete and rough wood (timber) have greater potential; however, potential for increasing tangential shear strength by increasing roughness is limited by the shear strength of the adjacent frozen soil. Creosoted wood, steel with a mill varnish or red lead or other coating substances have the least adfreeze bond potential.

(5) In tests in a permafrost area at Fairbanks, Alaska, average maximum adfreeze bond stresses as high as 60 psi have been measured on uncoated steel piles embedded in a silt, under natural freezing conditions. Since this is an average value over the area of adfreeze, higher unit values undoubtedly were developed in the coldest upper levels of the seasonal frost zone. Values decreased rapidly when the rate of advance of the freezing plane slowed, as stress relaxation occurred in creep. Thus, any design measure which results in slower frost penetration, and/or higher temperatures at the adfreeze bond surface and smaller bond area will lower peak adfreeze bond forces and increase probability of stability against frost heave. The same is not necessarily true of direct uplift as illustrated in figure 4-42a or of frost thrust as illustrated in figure 4-42b, as direct forces may remain high so long as ice segregation is occurring, even with a stationary position of the freezing plane.

(6) When a structure is permitted to "float" on the annual frost zone as in figure 4-42a, detrimental frost effects can be best avoided, first by minimizing the magnitudes of seasonal displacement through the heave reducing effect of a surcharge provided by a non-frostsusceptible granular mat and, second, by insuring uniformity of the remaining frost effects through (a) selection of sites with as nearly uniform soil conditions as possible, (b) careful control of thickness, soil characteristics, and drainage of the mat, (c) extending the mat a sufficient distance beyond the structure perimeter so that possible edge effects on footings are minimized, and (d) providing supplementary shading of the foundation if appropriate.

(7) A situation similar to that shown in figure 4-42a may develop inadvertently in temporarily exposed footing or foundations on frost-susceptible materials during construction if adequate protection against winter frost heaving is not provided. In two documented cases, at Anchorage, Alaska; and at a group of mid-western construction sites, erected but incomplete structures were bodily lifted as much as 4 to 6 inches by frost action before the heaving was noticed[148]. In the case of the mid-western installations tilting due to differential heaving also occurred. At both locations the foundation soils were allowed to thaw gradually and evenly; the foundations returned essentially to their design grades and the structures were completed successfully. In an inverse case, frost heave of various parts of an Alaskan schoolhouse was caused when artificial refrigeration

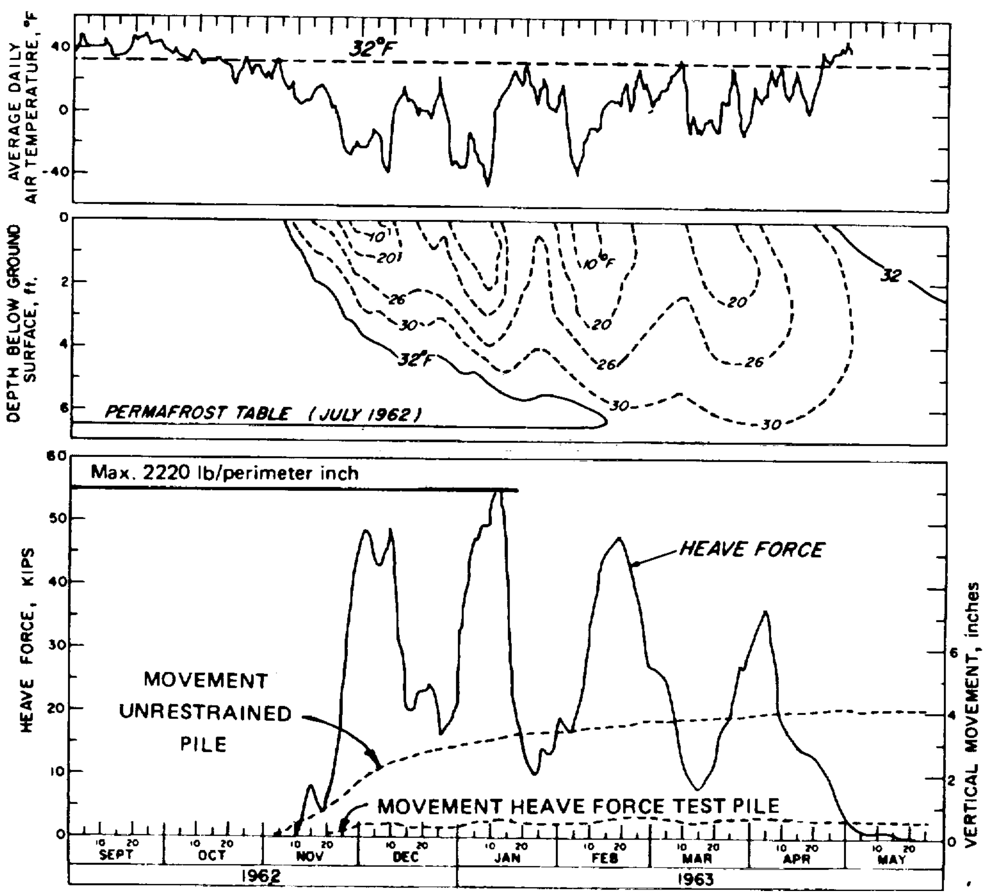

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Figure 4-44. Test observations, 1962-63, 8-inch steel pipe pile, placed with silt-water slurry in dry-augered hole[51]. "Heave force test pile" was restrained in order to obtain force measurements, permitting only the minimal movement shown. "Unrestrained pile[55] was an identical pile allowed to heave freely with no imposed vertical loading.

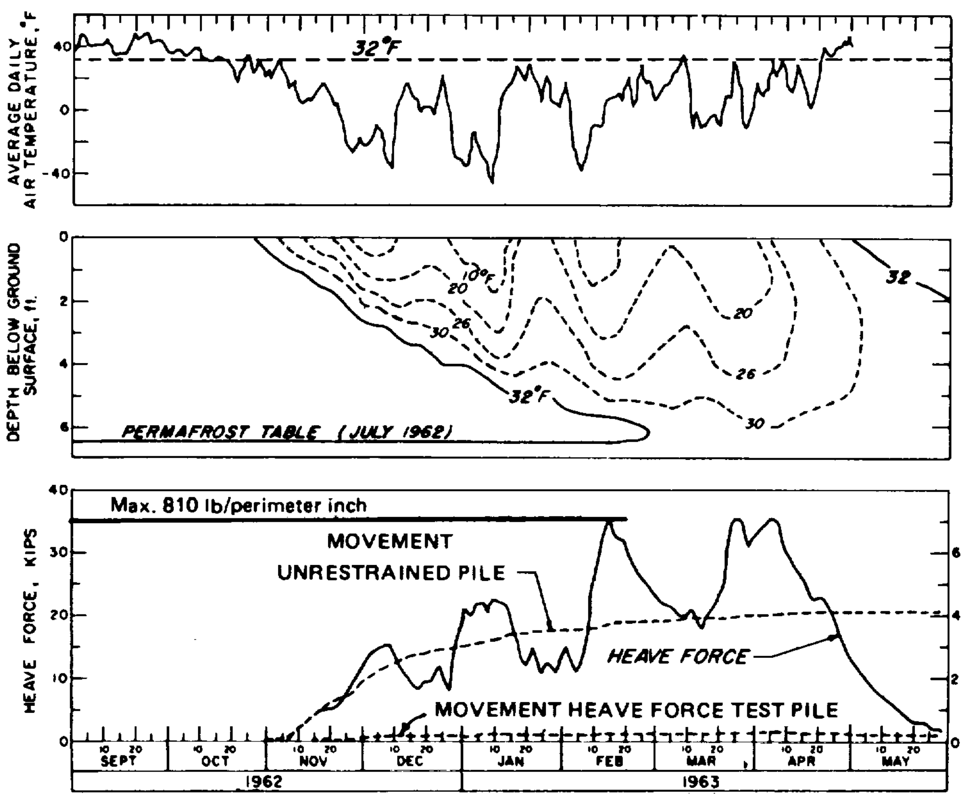

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Figure 4-45. Test observations, 1962-63, creosoted timber pile, average diameter. Heave force pile, 14 in.; unrestrained pile, 12 in. Placed with sill-water slurry in dry-augered hole[51]. "Heave force test pile" was restrained in order to obtain force measurements, permitting only the ,minimal movement shown. "Unrestrained pile" was an identical pile allowed to heave freely with no imposed vertical loading.

was employed to control degradation of permafrost under the structure[111].

(8) Foundations of the type shown in figure 4-42a are relatively immune to progressive frost action effects as the foundation readily returns each summer to its original position. However, foundations of the types illustrated in figures 4-42b and c are susceptible to possible progressive jacking by frost action. In these designs, irrecoverable deflections must be positively prevented.

(9) Where fixed type foundations are used as in figure 4-42c, detrimental frost action effects can be controlled by placing non-frost-susceptible soils in the annual frost zone to avoid frost heave problems; providing sufficient embedment or anchorage to resist movement under the heaving forces (sufficient integral strength must also be provided in foundation members to ensure such forces); providing sufficient loading on the foundation to counterbalance heaving forces; isolating foundation members from uplift forces by various means; or in seasonal frost areas by taking advantage of natural heat losses to minimize adfreeze and/or frost heave (here however, the possibility of future standby deactivation of the structure without heat must be considered).

(10) A non-frost-susceptible foundation mat or backfill can be of substantial help in achieving desired control. In addition to providing material which in itself will not impart frost heaving forces to the foundation, the mat will impose a surcharge on the underlying soils to reduce frost heaving and provide a useful thermal barrier which will avoid the extremely low temperatures in the potentially adhering frost-susceptible underlying soils. Maintaining warmer (though still below freezing) temperatures decreases the adfreeze strength of the insitu soils.

(11) All forces which might distort the supported structure must be carefully anticipated. As discussed elsewhere some types of foundations are self-adjusting for non-uniformities of frost heave.

(12) Thermal piles offer increased anchorage against frost heaving through lowering of pile surface temperatures in permafrost. Self-refrigerating thermal piles also tend to reduce depth of summer thaw and furnish heat to the annual frost zone during the period of frost heaving, thereby reducing the adfreeze bond stress in the zone of frost jacking.

(13) Various methods of isolation are available to reduce the extremely high adfreeze strength and upward forces imposed on piling, footings and other foundations by frost heaving.

(14) Heave force isolation of piles is required when there is insufficient length of pile in permafrost, particularly when warm permafrost is present. This condition is particularly common when bedrock or another bearing stratum exists at relatively shallow depths. The bearing stratum can competently support the live and dead loads on a pile but cannot provide sufficient anchorage to resist frost heaving forces unless the pile is anchored at considerable additional expense. Such conditions are quite common at bridges, where the thermal regime of the permafrost is influenced by the stream. In other situations, such as piles or poles which are lightly loaded or carry only transient loads, the depth of embedment in permafrost required solely to resist the frost heave forces may produce uneconomical design. Isolation may also be more economical in seasonal frost areas.

(15) Heave force isolation may be accomplished by casing the pile or foundation member or backfilling around the foundation member with treated soil. The annulus between pile and casing is normally filled with an oil-wax mixture which has a thick consistency, as shown in figure 4-46a. Simple casings within the annual frost zone normally will be jacked progressively out of the ground by frost action. Therefore, plates or flanges should be employed at the bottom of the casings to resist casing heave.

(16) To avoid the difficulties and costs involved with casing, a premixed backfill of soil, oil and wax may be used to reduce frost heave thrust on the upper sections of the pile to acceptably low values, as shown in figure 4-46b. This method of heave isolation offers a somewhat greater lateral pile support than the casing and oil-wax method shown in figure 4-46a.

(17) Coating the pile length in the annual frost zone with low friction material, as well as creosoting", may temporarily reduce frost heaving but must not be relied on for this purpose in permanent construction. Chemical additives have also been added to the active zone immediately around piles but were not found to be significant or of sufficient life to reduce heaving. Various methods of providing additional shear strength or anchorage in permafrost have been studied to combat frost heaving of lightly loaded piles but were found to be only partially effective. Some of the methods investigated including notching the pile, driving railroad spikes in timber piles, welding angle iron on flanges of steel piles and providing plates on the base of various pile types.

(18) While a large surface area in permafrost is desired to provide a greater capacity, the section of pile passing through the active layer should be as small as possible. Changes in pile cross sections or surface areas available may be accomplished by the use of composite piles. Care should be taken in using composite piles to ensure that the pile has adequate strength to resist tension. Reducing the surface area of timber piles in the annual frost zone may be accomplished by placing timber piles butt down. In addition to a small cross section in

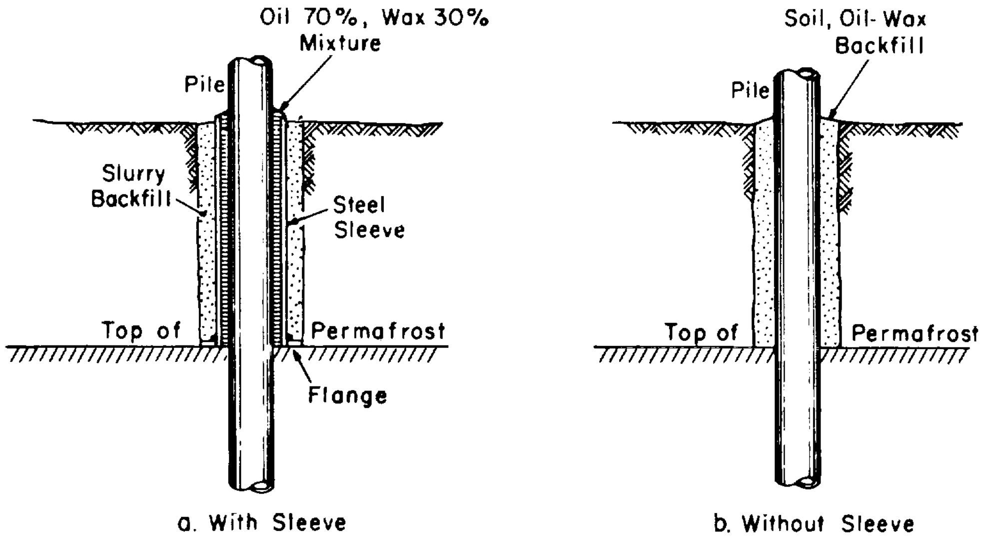

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Figure 4-46. Heave isolation (by CRREL).

the annual frost zone, timber piles placed in this manner have a preferred batter to further reduce frost thrust together with improved anchorage.

4-4. Allowable design stresses on basis of ultimate strength. Ultimate strength and deformation characteristics of any particular saturated frozen soil depend primarily upon its temperature relative to 32 ° and the period of time that the soil will be subjected to a given stress. The ultimate strength increases as the temperature decreases.

a. There are two primary reasons for the increase in strength: first, the solidification by freezing of an increasing proportion of the water in the voids as the temperature drops (this is of importance primarily in fine-grained soils, especially in the temperature range between 32°F and 0°F); and secondly, the increase in strength of the ice fraction with decrease in temperature. In addition, there is the possibility of a contribution resulting from soil matrix consolidation due to ice segregation",'". The ultimate strength of frozen soil also decreases with increase in the length of time over which a constant stress must be resisted. The dependence of ultimate strength of frozen soil on temperature and time-duration of constant stress application is shown in a plot of unconfined compressive strength vs. temperature for a saturated frozen fine sand (fig. 4-47)[87]. It should be noted that for a given reduction of temperature, the increase in long-term strength is smaller in magnitude than the increase in short-term strength; however, the proportionate increases are comparable, as shown in figure 4-48.

b. The decrease in strength with increase in time over which a constant stress must be resisted is further illustrated in the unconfined compression test results shown in figures 4-49 and 4-50 for frozen saturated Ottawa sand. The family of experimental curves in figure 4-49 show creep curves for different applied constant stresses at 29°F. Proceeding from high to the lower stresses, the slopes of the curves decrease until the lower curve approaches a nearly horizontal straight line. The latter curve, which corresponds to the maximum stress that the soil can resist indefinitely, is considered the long-term strength of the soil.

c. The plot of time to ultimate failure vs. ultimate strength (fig. 4-50) indicates that for this particular frozen soil the long-term strength at 25 °F is in the order of about 20 percent of that determined from a standard test for unconfined compressive strength. Tests on silts

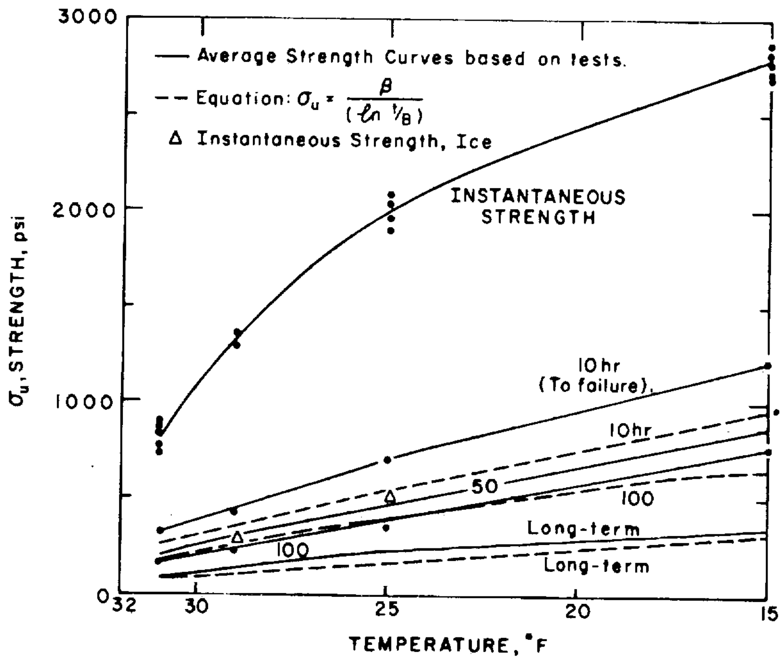

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Figure 4-47. Frozen soil creep tests, unconfined compression, Manchester fine sand[87]. Instantaneous strength is maximum stress determined by loading specimen at a constant strain rate of 0.033/min. Long-term strength is maximum stress that the frozen soil can withstand indefinitely and exhibit either a zero or continually decreasing strain rate with time.

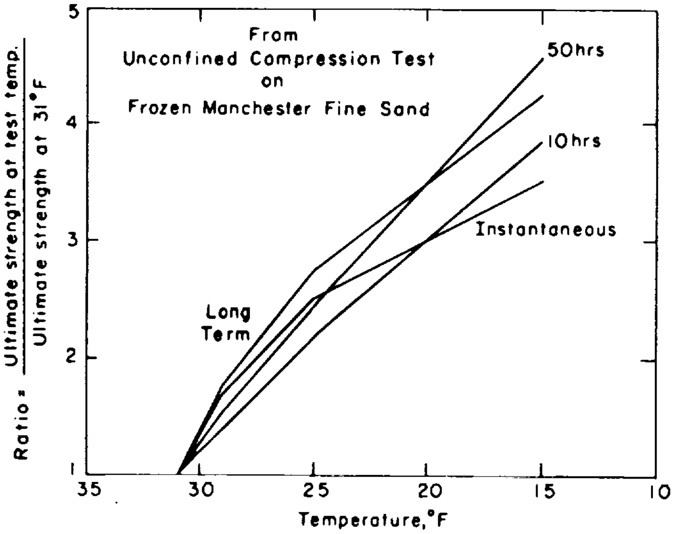

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Figure 4-48. Variation of ultimate strength ratios with temperature for various rates of loading (computed from data in fig 4-47).

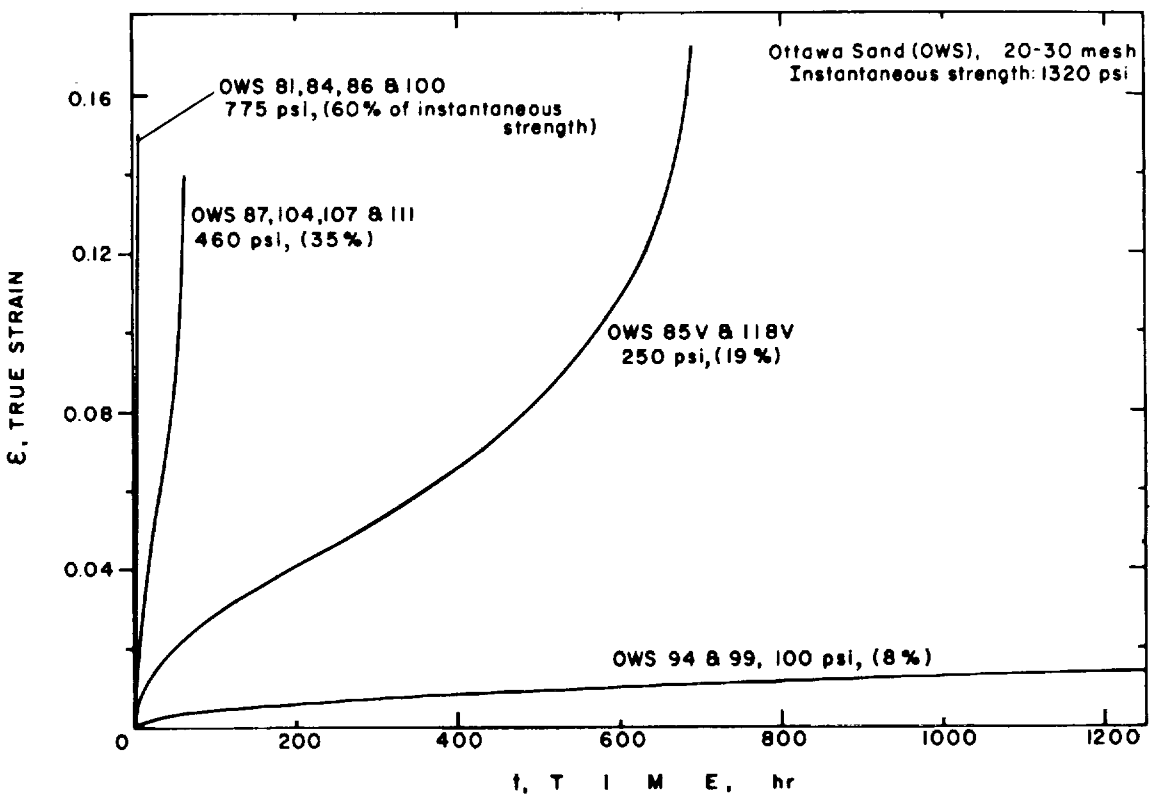

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Figure 4-49. Unconfined compression creep curves for frozen Ottawa sand at 29°F [87].

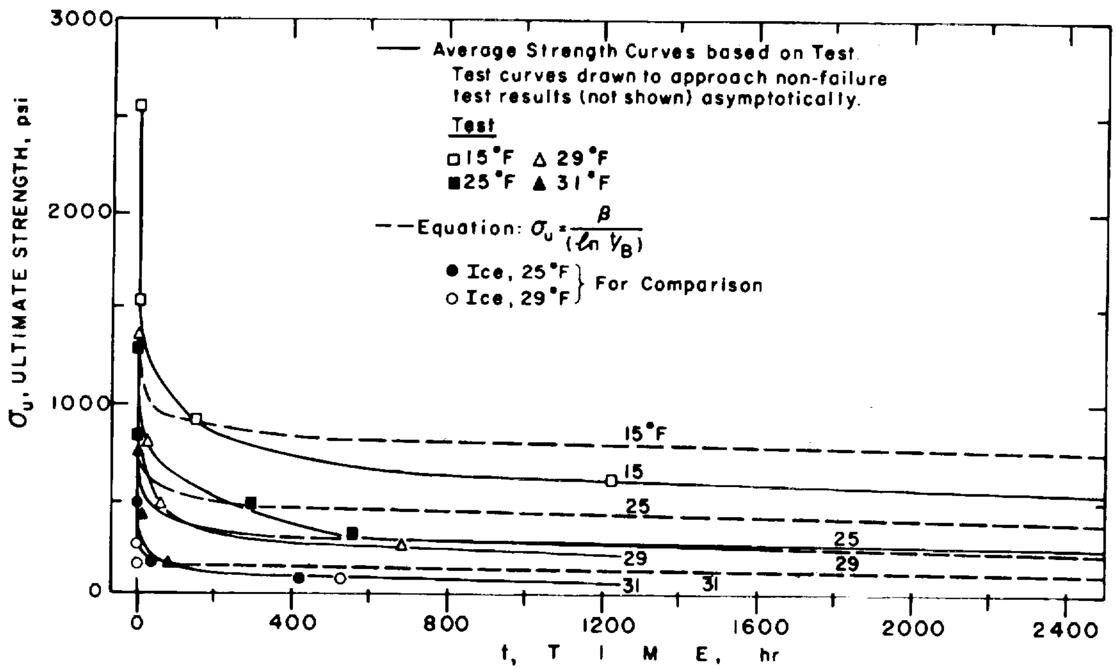

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Figure 4-50. Ultimate strength vs. time to failure for Ottawa sand (20-30 mesh) at various temperatures, comparing test data with computed values[87].

and clays indicate that the long-term strength level can be as low as 5 to 10 percent of that determined from a standard test for compressive strength.

d. The load carrying capacity of frozen soil is essentially determined by its shear strength and within certain limits the relationship between shear strength and normal pressure may be written as:

(Equation 2) $s = c + p \tan \phi$

where c = cohesion, which is dependent upon both temperature and time

ϕ = angle of internal friction

p = total normal stress.

e. To show graphically how shear strength varies with time, Mohr's envelopes for frozen sand are shown in figure 4-51. The envelopes are plots of the ultimate shear strength vs. normal stress for different rates of applied strain in controlled strain rate type triaxial tests. Two characteristics are of interest here. First, the ultimate shear strength is less for the low strain rates than for the higher ones; and secondly, the envelopes are noticeably curved, that is, shear strength is not directly proportional to normal stress. Therefore, when testing frozen soil for shear strength, the tests must be performed using a normal stress near the value to which the in-situ material will be subjected and large extrapolations of the shear envelopes should be avoided. It is permissible and conservative to use a secant to the envelope if a straight line relationship between the shear and normal stress is desired, provided the secant intersects the envelope at the in-situ stress value.

f. Experimental work indicates that long-term shear strength of saturated frozen soils can be determined for practical purposes by the equation:

$$\sigma_t = \frac{\beta}{1n(t/B)} \quad \text{(by Vialov)} \qquad \text{(Equation 3)}$$

where σ_t = the constant stress level at which failure will occur at time t in psi

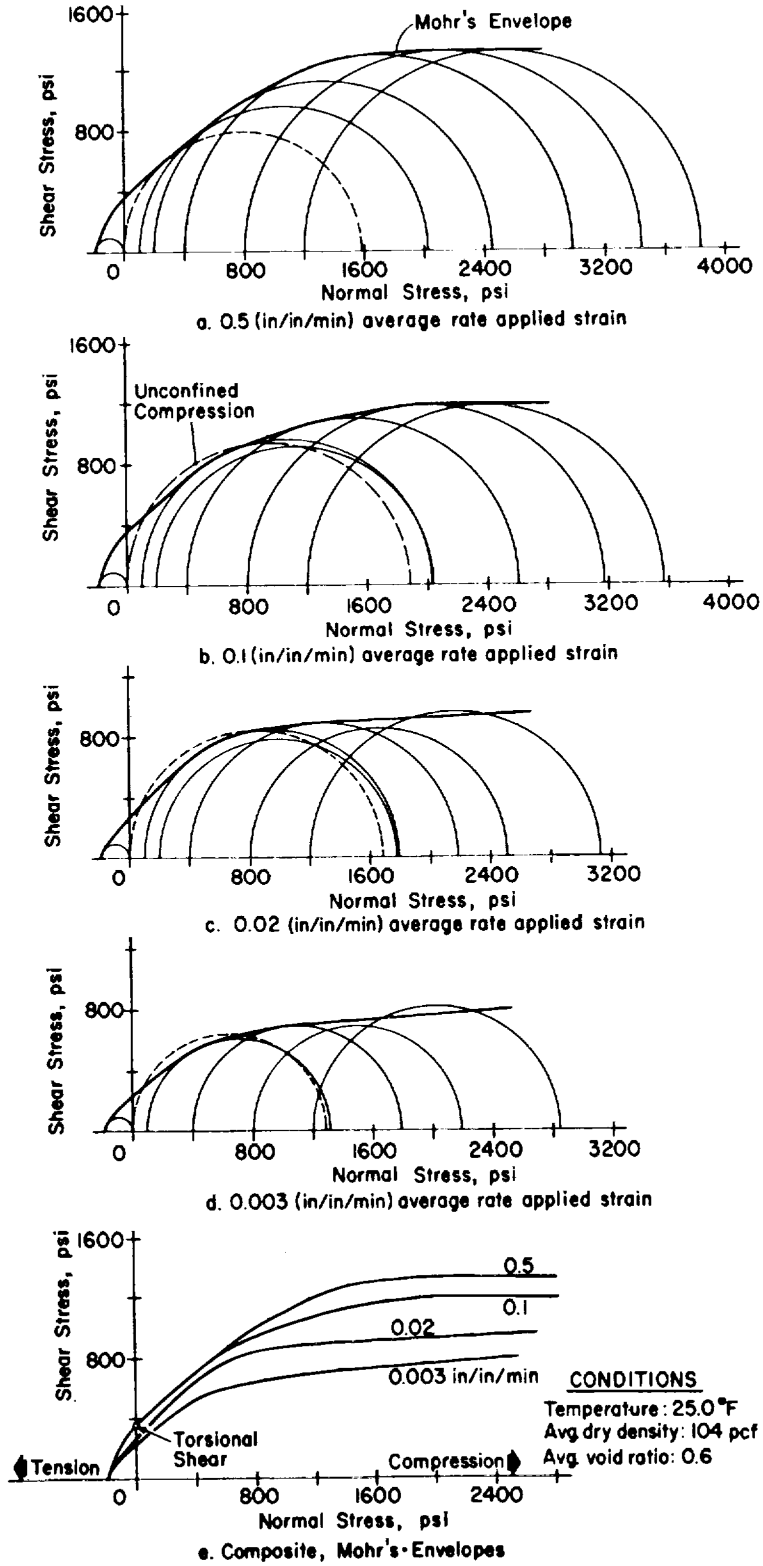

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Figure 4-51. Mohr's envelopes for frozen sand (Ottawa sand, 20 - 30 mesh).

t = period of time after application of stress (σ_t) that failure will occur, hour

β and B are soil constants that are temperature dependent.

g. It should be noted that equation 3 is an empirical relationship for creep strength and it is not valid for small values of t since the strength would become infinite when time approaches zero. For practical applications of equation 3, the value of t should not be less than, say, one minute.

h. One method of determining the ultimate bearing capacity of a foundation on permafrost is to assume that frozen soil is a purely cohesive material and use an appropriate foundation bearing capacity equation.

i. This assumption is conservative since the internal friction term (p tan 0 of equation 2) is assumed to be zero. Internal friction can contribute substantial shear resistance in some frozen soils, such as unsaturated frozen soils; however, the determination of the value of < requires that triaxial creep tests or similar tests be performed for the specific in-situ conditions under consideration, whereas a value for the cohesion factor can be determined by relatively simple unconfined compression creep tests using procedures described below. For footings, the equations developed by L. Prandtl and K. Terzaghi can be applied using a cohesion value thus determined. This method is particularly appropriate for frozen silts and clays and can be readily applied also to frozen fine-to medium-grained saturated sands. For frozen gravels and tills somewhat greater difficulty is involved.

j. To determine the strength value of the frozen soils for use in equation 3, a minimum of two unconfined compression creep test are required on undisturbed soil samples of each type of foundation soil. These tests must be conducted near the estimated temperature or maximum seasonal temperature (critical temperature) of the natural foundation soil. To expedite the creep testing, one of the creep tests should be performed at a stress level of about 60 percent of the conventional unconfined compressive strength and a second test near the 40 percent strength at the critical temperature. In general, the test at the 60 percent stress level should require less than eight hours to perform; the test at 40 percent stress level may require as much as three days. Using the period of time required for each of the two test samples to fail and the corresponding applied creep stress for each test, the constants B and B can be evaluated by substituting twice into equation 3 and solving the simultaneous equations. By substituting the estimated design life of the structure into equation 3 (unless specific justification exists for assuming a different life span, a 25-year life should be used for a permanent structure), a value of σult is determined. A nomogramt[6] to solve equation 3 is shown in figure 4-52 with an illustration of its use and may be used as a check on these computations. The nomogram is limited in its application to situations where the shortest duration of test is longer than one hour. Since the cohesive strength is about one-half the unconfined compressive strength, the value of ultimate cohesive strength is one-half the σult so determined. The value of ultimate cohesive strength must still be reduced by an appropriate factor of safety. Factors recommended in TM 5-818-1/AFM-88-3, Chapter 7 [5] should be used.

k. Allowable long-term cohesive stresses for a few specific frozen soils are listed in table 4-4. These stresses include a factor of safety of 2.0. The degrees of reduction involved in these stress values from short-term strength results may be visualized in figure 4-53 which shows Mohr's envelopes for six frozen mineral soils, frozen peat and ice, based on tension and unconfined compression tests. The 1 T/ft^2 allowable design stress for Manchester fine sand at 31.0 °F is indicated on the ordinate of the left hand diagram and may be compared with the envelope for the same material. The 2.0 T/ft2 allowable design stress for the same soil at 29° F is shown on the right hand diagram. It is emphasized that values in table 4-4 are given for general guidance and that cohesive stress values for the specific foundation soil under consideration should be determined by test procedures previously outlined.

l. When foundations are supported on soils containing large size particles, such as gravel or glacial till, the working design strength will depend on the amount of ice present. If there is a substantial amount of segregated or excess ice, the behavior of the material should be assumed to correspond to that of the frozen fine fractions of the soil, or of ice. If the soils are under-saturated or the voids are completely filled but there is no excess ice and full particle to particle contact exists, the soils may be expected to behave like unfrozen soils with normal void ratio at relatively low stress levels. These stress levels do not disturb essentially the original particle to particle bridging, which the ice here helps to maintain. However, creep behavior should be expected at higher stress levels which involve sufficient strain deformation to force the ice matrix into positive response. It should be kept in mind that in frozen soils the volume changes which in unfrozen soils attend shearing deformation are essentially prevented, thus greatly modifying shear response.

4-5. Estimation of creep deformation.

a. Determination of the bearing capacity of frozen foundation material on the basis of ultimate strength as described in the preceding section will not necessarily insure satisfactory performance, because unacceptable progressive creep deformation may occur. As described

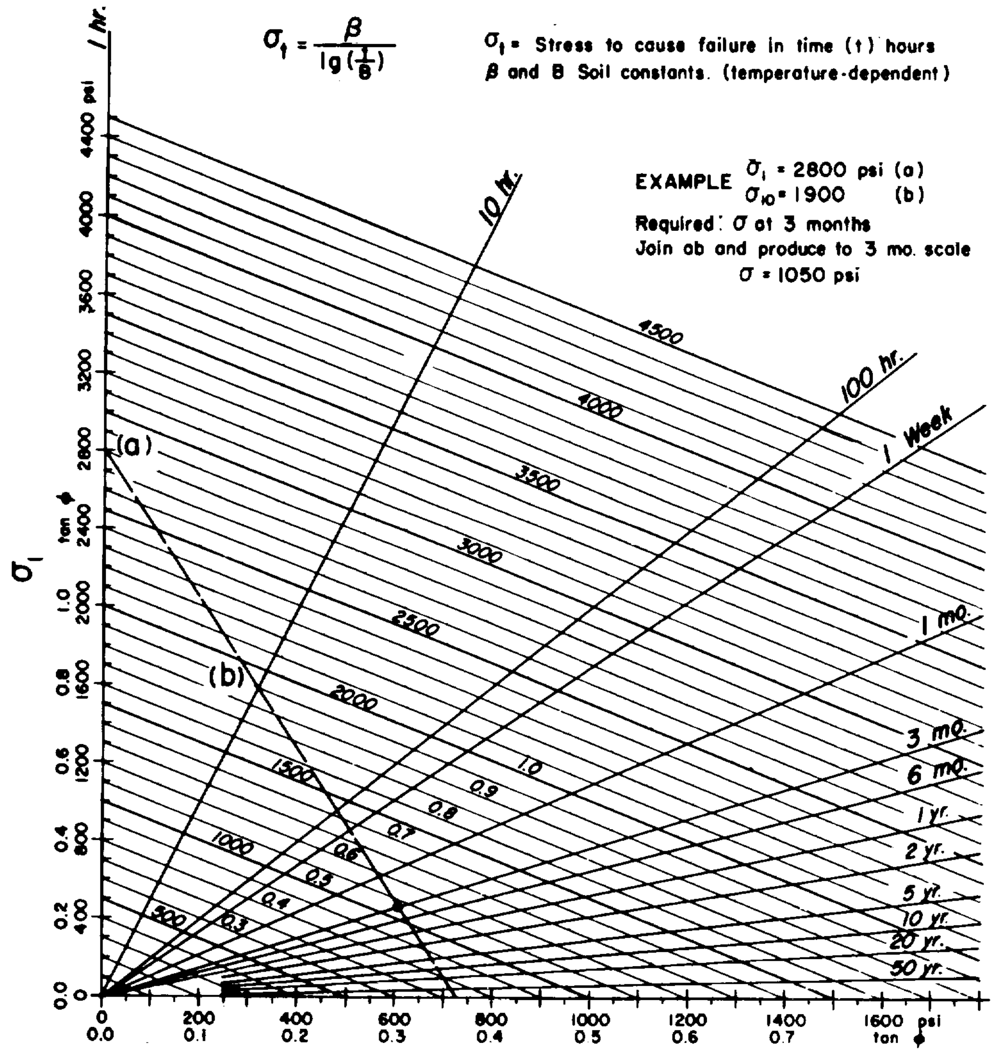

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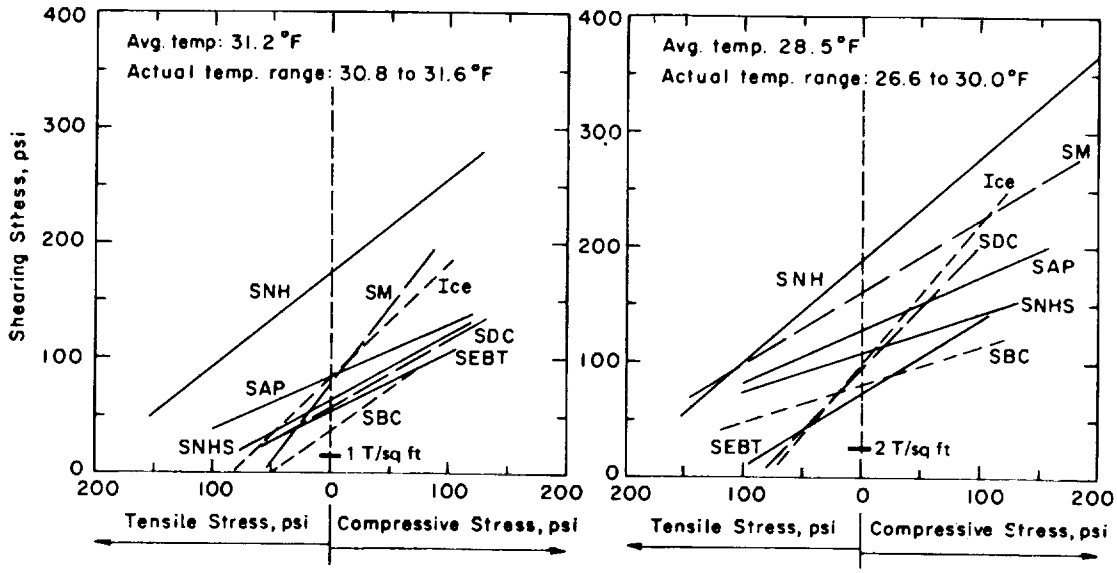
Figure 4-52. Stress and time to failure in creep[186].

Table 4-4. Allowable Design Cohesive Stress for Saturated Frozen Soil in T/ft^2 on Basis of Ultimate Strength.

The values for stress shown include a safety factor of 2.0. The stress values should only be used where cohesion is the sole factor involved.

U. S. Army Corps of Engineers Frozen Soil	Critical (highest) temperature of frozen soil beneath base of the foundation			
	310°F	290°F	25°F	150°F
Saturated Ottawa sand [87] (20-30)	1.0	2.5	4.5	10.0
Manchester fine sand[87] (Saturated uniform fine sand)	1.0	2.0	3.5	6.0
Hanover silt (Saturated silt, ML, non-plastic, no visible ice lenses, w ≤ 35%) (Unpublished CRREL data) -	0.6	1.5	3.0	5.5
Suffield clay (Saturated clay, CL; LL 35, PL 20; no visible ice lenses, w < 40%) (Unpublished CRREL data)	0.5	1.0	2.5	5.0



1. Gradations and other characteristics of these soils are shown in figure 2-11.
2. Rate of stress increase in unconfined compression tests, 400 psi/min. Rate of stress increase in tension tests, 40 psi/min.
3. Insufficient data available to show probable curvature of envelopes.
4. Average degree of saturation:

SNH, Manchester Fine Sand, 84%
SM, McNamara Concrete Sand, 89%
SNHS, New Hampshire Silt, 88%
SEBT, East Boston Till, 93%
SAP, Alaskan Peat, 98%
SBC, Boston Blue Clay, 98%
SDC, Dow Field Clay, 98%

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Figure 4-53. Mohr envelopes for frozen soils under moderately rapid loading, from tension and unconfined compression tests[33].

in paragraphs 2-5a and 4-4 and as illustrated in figures 2-15, 4-49 and 4-50, frozen soils and ice exhibit creep characteristics under long term loads down to at least as low as 5 to 10 percent of their rupture strengths under relatively rapid loading. When slow, progressive movement occurs in foundations on saturated frozen soil in absence of thawing, it is generally the result of creep. Creep is a time dependent shear phenomenon in which the total volume of the stressed material remains constant; i.e. the stressed soil flows rather than consolidates. In TM 5-852-1/AFM 88-19, Chapter 1, slope creep is defined as "extremely slow downslope movement of surficial soil or rock debris, usually imperceptible except by long-term observation." In that case the movement usually involves freeze or thaw action in strata near the surface and downslope movement of seasonally thawed soil, together with creep of frozen materials when stress and temperature conditions favor this. For creep of frozen material, the ice filling the soil voids may be considered to be a fluid of extremely high viscosity. Within normal pressure and time frame, consolidation of soil can only occur if air or other gas voids are present in the soil mass or if part of the soil moisture is not frozen.

b. In general, present design practice is to avoid the problem of creep in frozen soil foundations either by supporting footings on mats of well drained non-frostsusceptible gravel or other material which spread stresses sufficiently so that stresses on underlying confined frozen materials are conservatively low, or by placing foundations at a sufficient depth in the ground so that the overburden pressure effectively minimizes foundation-induced creep.

c. When analysis indicates that a footing, raft, pier or similar foundation designed on the basis of ultimate strength with recommended factor of safety will develop unacceptable creep deformation over the life of the facility, the design must be revised to bring the deformation within acceptable limits.

d. Various empirical equations have been proposed for the prediction of creep of frozen soil in unconfined compression. At the present time, these equations do not take into account the complex stress and deformation conditions of the soil beneath a foundation. For the first approximation of the amount of creep that may be expected, the following empirical equation similar to Vialov[107] may be applied to a foundation:

$$\text{strain} = E = \left[\frac{\sigma t^{\lambda}}{w(0+1)^{k}}\right]^{1/m} + E_o \qquad \text{(Equation 4)}$$

where:

- σ = stress in the material under consideration, psi
- t = time that stress is to act, hr
- θ = number of degrees below the freezing point of water, °F
- ε_o = strain that occurs immediately upon application of stress (this term can be neglected for the purpose of estimating creep)
- m, λ, w, k = constants that depend on properties of material.

Typical values for m, λ, w, and k for equation 4 are given for specific soils in table 4-5. These values were derived empirically by laboratory tests demanding quite precise measurements of the absolute values of strain and requiring several tests for evaluation of the constants. Care must be taken to use the proper units consistent with those given in the table.

e. A conservative method for estimating the vertical creep of a foundation is to assume that the foundation is supported by a column of frozen soil having a height equal to 1/2 times the least plan dimension of the foundation and apply equation 4 to compute the strain and hence the deformation of the soil column. In applying equation 4, values for stress and temperature are assumed to be constant for the period of time under consideration. The average temperature of the column of permafrost for the critical period of the year can be projected from ground temperature records or from on-the-spot temperature measurements. The critical period of the year is that time of year when permafrost temperatures beneath the foundation are warmest. The magnitude and distribution of stress under the foundation can be approximated by using elastic theory, as outlined TM 5-818-1/AFM 88-3, Chapter 7[5]. Since magnitude of stress decreases with depth, it is necessary to use an equivalent constant stress in order to apply equation 4. A closer approximation of the amount of creep can be obtained by dividing the soil beneath the foundation into an arbitrary number of horizontal zones and using an average constant stress and temperature for each zone. Using these average values and the thickness of each zone, equation 4 can be used to estimate the vertical deformation of each zone. The total deformation will be the sum of the deformations of all the zones. Where the soil is stratified, the boundaries of some of the zones should be coincident with the stratum interfaces.

f. It is emphasized that this procedure will give only an order of magnitude of the amount of creep and the constants in table 4-5 apply only for the specific soils listed and are given only as a guide.

g. A second, more accurate, method of predicting creep requires performance of unconfined compression creep tests on undisturbed samples of the foundation soil at the design stress level and at the predicted temperatures of the foundation soil and application of the following empirical equations:

Table 4-5. Constants for Equation 4 (by CRREL and ref 107)

All constants are dimensionless except that the units of w for the equation to be dimensionally correct are: $[\text{psi(hr)}^{\lambda}]/{}^{o}\text{F}^{k}$.

Frozen Soil	m	λ	ω	k
Saturated Ottawa sand* (20-30 mesh)	0.78	0.35	5500	0.97
Manchester fine sand* (saturated uniform fine sand, 40-200 mesh)	0.38	0.24	285	0.97
Suffield clay* (saturated clay*, CL, LL 35, PL 20, no visible ice lenses, w < 40%)	0.42	0.14	93	1.0
Hanover silt* (saturated silt, ML, non-plastic, no visible ice lenses, w < 35%)	0.49	0.074	570	0.76
Callovian sandy loam** (sandy silt, ML)	0.27	0.10	90	0.89
Bat-baioss clay**	0.4	0.18	130	0.97

*Data from laboratory tests, CRREL.
**Data from Vialov et al. (1962), Ch. V.[107]

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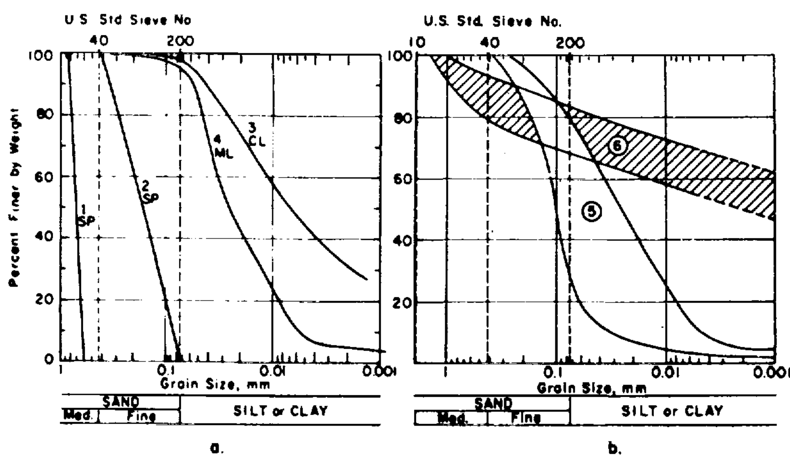

No	Description	Nat w %	LL	PL	PI
1	Ottawa SAND (20-30)	—	—	—	—
2	Manchester fine SAND	—	—	—	—
3	Suffield CLAY	—	35	20	15
4	Hanover SILT	31	Non-plastic		
5	Callovian Sandy LOAM	≈26	25-36	21-30	4-7
6	Bat Baioss	20-24	23-58	14-21	9-31

$$\frac{1}{t} = \left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_1}\right)^M \text{ or } \dot{\varepsilon} = \dot{\varepsilon}_1 t^{-1/M} \qquad \text{(Equation 5)}$$

By integration

$$\varepsilon = \dot{\varepsilon}_1 \left(\frac{M}{M-1}\right)\left(t^{\frac{M-1}{M}} - 1\right) + \varepsilon_o \qquad \text{(Equation 6)}$$

when M is greater than zero and not equal to one,

or, $\varepsilon = \dot{\varepsilon}_1 \ln t + \varepsilon_o$ when M is equal to one,

where

ϵ = strain total

$\dot{\epsilon}$ = strain rate

$\dot{\epsilon}_1$ = strain rate 1 hour after stress is applied

ϵ_o = initial strain that occurs at the time of load application. ϵ_o can be neglected for the purpose of evaluating creep.

M = slope of the log 1/t vs log strain rate plot

An illustration of the use of this method is given in paragraph 4-7*b*(2).

h. The third and probably most accurate method of predicting creep is to run a field test on a prototype or large size model of the foundation under consideration and apply equation 6. The field test should be performed on the model using the same configuration, soil pressure and soil temperatures as for the foundation to be constructed, and on the same frozen soil. The design stress should be applied to the model as nearly instantaneously as possible but without impact. (One method of applying the load is to release the hydraulic pressure from jacks in a manner so as to quickly transfer a dead weight load from the jacks to the model foundation.) After the full load is applied, the deformation of the model should be recorded at frequent intervals to define the time vs. deformation curve for a period of eight hours. The elevation should be recorded before loading and immediately after loading. (The difference between these two readings gives an estimate of the instantaneous deformation that occurs during load, i.e. eo.) Using the time after load application as time zero, then deformation readings should be taken at times 1, 2, 3, 4, 5, 10, 20, 30 minutes, 1 hour, and every hour, until 8 hours have elapsed.

i. Using the data obtained from the model tests, the values of e, and M in equation 6 can be determined graphically. One technique is to use the slopes of the tangents to the deformation vs. time curve on arithmetic coordinates at times of 1/2 hour and 1 hour after stress application (see figure 4-54) for details). A second technique is to determine the rate of deformation of the foundation at several times and plot log e vs. log l/t curve for the 8 hours of the test. The slope of this curve is the value of M. The value of ε_1 can be read from the curve directly, i.e., at 1 hour; see figure 4-55 for details. The foundation soil temperature is nearly constant and is the same for the model as for the foundation during the critical period of the year (defined above). The test is performed in in-situ soil that the structure foundation is to be founded on and that the same soil conditions extend at least to a depth equal to the smaller dimension of the entire foundation. The model dimensions are large enough to minimize the edge and side effects.

k. The various test requirements may make this approach difficult to employ; the soil temperature requirements, for example, may substantially restrict the time of year within which the field tests may be performed.

l. It must be emphasized that the methods and ideas presented here for predicting creep of frozen soils are still under investigation.

m. Where the tolerable foundation movement is very small, a special investigation for the determination of creep deformation may be required.

4-6. Dynamic loading.

a. General.

(1) Foundations supported on frozen ground, ice or snow may be affected by high stress type dynamic loadings such as shock loadings from high yield explosions, by lower stress pulse type loadings as from earthquakes or impacts, or by relatively low stress, relatively low frequency, steady-state vibrations. In general, the same design procedures used for non-frozen soil conditions are applicable to frozen soils. Design criteria are given in TM 5-809-10/AFM 88-3, Chapter 13[3], TM 5-856-4[18], and EM 1110-345-310[20]. These manuals also contain references to sources of data on the general behavior and properties of non-frozen soils under dynamic load and discuss types of laboratory and field tests available. However, design criteria, test techniques and methods of analysis are not yet firmly established for engineering problems of dynamic loading of foundations. Therefore, HQDA (DAEN-ECE-G), WASH DC 20314 or HQUSAF/PREE, WASH, DC should be notified upon initiation of design and should participate in establishing criteria and approach and in planning field and/or laboratory tests.

(2) All design approaches require knowledge of the response characteristics of the foundation materials, frozen or non-frozen, under the particular load involved. As dynamic loadings occur in a range of stresses, frequencies and types (shock, pulse, steady-state vibrations, etc.) and the response of the soil varies depending upon the load characteristics, the required data must be obtained from tests that produce the same responses as the actual load. Different design criteria are used for the

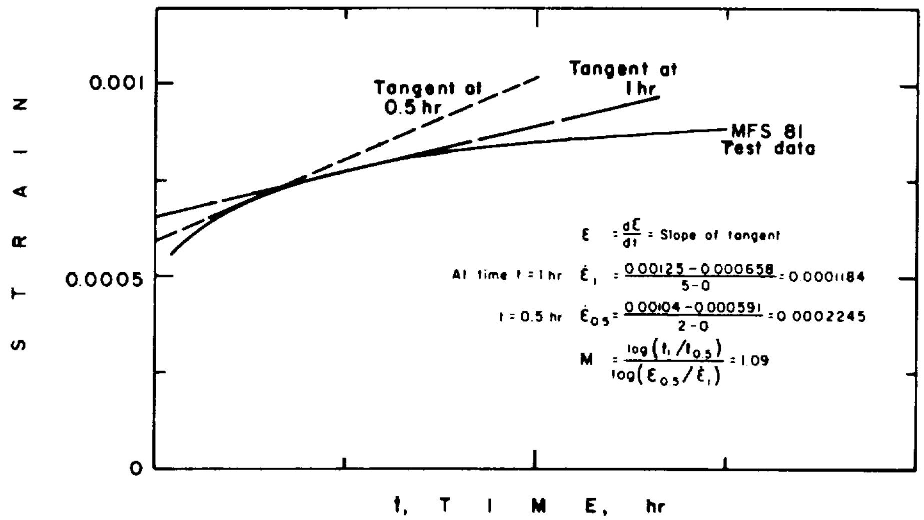

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Figure 4-54. Frozen soil creep test on Manchester fine sand, unconfined compression, 200 psi at 15°F [87].

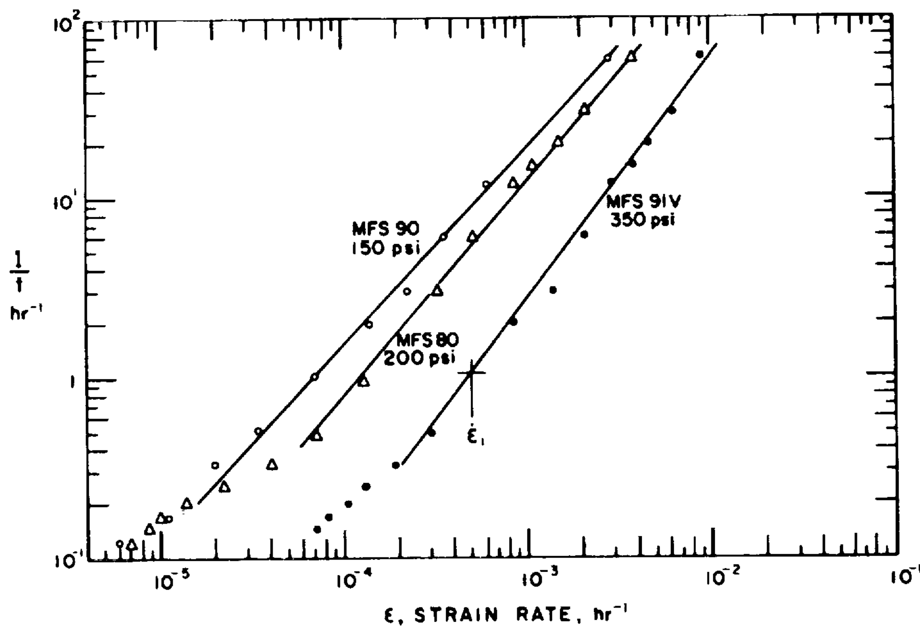

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Figure 4-55. Frozen soil creep tests on Manchester fine sand, unconfined compression, 15°F [87].

different types of dynamic loading and different parameters are required. Such properties as module, damping ability, and velocity of propagation vary significantly with such factors as dynamic stress, strain, frequency, temperature, and soil type and condition.

b. Determination of response characteristics of foundation materials. The testing of frozen materials under dynamic loading has been only recently explored and relatively few published data are available. Some data are shown in figure 2-17. It will usually be necessary to conduct a test program for the particular site and the particular soils involved.

(1) In-situ tests. Two methods are available. In a procedure described in a Waterways Experiment Station paper a vibrator is placed on the surface and operated at a range of frequencies[106]. The characteristics of the wave are measured, yielding a relationship between shear modulus and depth. Various seismic procedures may also be used[84,124,75].

(2) Laboratory tests. Few laboratories are presently equipped to test frozen soils under dynamic loads, but suitable techniques are available. Foundation analysis for high stress, shock type loads requires knowledge of the equation of state for the condition of interest so that the conservation of energy laws may be applied. Test techniques yielding pressure-volume-temperature relationships for frozen soils have been described in several papers[32,63,128,170,176]. Design analysis for low stress, steady-state vibration type loading requires values for deformation moduli, velocity of wave propagation, and internal damping for the particular soil. Kaplar[68] and Stevens[93] have described two test techniques. The first of these techniques yields moduli of elasticity and rigidity, longitudinal and torsional velocities of wave propagation, and Poisson's ratio. The second technique uses viscoelastic theory and yields complex Young's moduli, dilatational and shear velocities and internal damping factor expressed as the tangent of the lag angle between stress and strain[93,94]. The latter value may be expressed as an attenuation coefficient. (Consider a plane wave passing through a solid. If the displacement amplitude at a distance from the source is A_{19} and at a distance, X_1 farther along is A_{25} then: $A_2 = A$, e^{-ax}, and α is the attenuation coefficient. It is a property of the material.) If damping is small as it usually is in frozen soils, the complex modulus does not differ significantly from the elastic modulus.

c. The response of frozen materials to dynamic loads. In general, frozen soils are more brittle, are stiffer (that is, have higher moduli) and have less damping capacity than non-frozen soils. However, these properties vary widely, primarily with temperature, with ice volume/s oil volume ratio, soil type, load characteristics, and degrees of ice saturation and segregation. The classification system for frozen soils describes frozen soils in terms of the most fundamental of these parameters. Rock also tends to be stiffer when frozen. Possibly both non-frozen and frozen materials may be present in a foundation, complicating the problem.

(1) *High stress dynamic loads.* While high stress loads may result from a variety of causes, most available design criteria have been developed for the case of protection of a structure against the shock loadings imposed by explosions. Design of the structure, including the foundation, for stresses resulting from explosions is covered in TM 5-856-4[18]. In general, the pressures resulting from an air blast are more critical to surface facilities than ground transmitted shock waves, and the question of the response of the soil does then not particularly enter the problem. However, if the structure is underground or the shock source is in the ground, then consideration must be given to the characteristics of the stress wave propagating through the ground and, in the cold regions, through frozen materials. The theory employed and general approach to the problem are given in TM 5-856-4. If the stress involved is sufficiently low or such that a change in state of the material does not occur, the required soil properties may be obtained as discussed in (3) below. If a change in-state does occur, as is possible in shock type loads, then the pressure-density-temperature relationship for the particular material must be obtained[32,63,67,90,121,176.]

(2) Dynamic loads imposed by earthquakes.

(a) TM 5-809-10/AFM 88-3, Chapter 13 [3], presents criteria for design of structures against earthquake damage, including earthquake intensities for design purposes for the state of Alaska and some other cold regions locations. Recent suggested procedures for earthquake design employ response spectrum techniques wherein the response of the structure in each mode is considered and total response is obtained by combining the separate modal responses. An example of the application of this technique has been presented by Severn and Taylor[190].

(b) All design techniques employed for nonfrozen soil conditions are applicable to frozen soil conditions, but the response of frozen soils to earthquake load may obviously be quite different. Of primary concern is the brittleness, greater stiffness and overall rocklike behavior of frozen soil as compared with nonfrozen soil. Stress wave velocities are much higher and damping is generally lower in the frozen soil. Propagation of the stress wave through permafrost may be faster and of higher intensity than for non-frozen soils.

(c) The U.S. Geological Survey has reported observations, on the Alaskan earthquake of 1964 [201]. Seasonally frozen soil on the surface acted as a more or less rigid blanket over the underlying non-frozen soil. Where the blanket was 2 or 3 feet thick, cracks of a brittle nature occurred, sometimes forming large slabs of frozen soil. In some cases a pumping action resulted, wherein non-frozen soil and debris were ejected through the cracks in the frozen soil layer. There was mention, in a few oases, of sliding occurring along the interface of the frozen and non-frozen layers.

(d) The design engineer should visualize the possible effects of an earthquake on the foundation, whether it may involve sliding of slabs of frozen soil in winter, sliding of saturated thawed soil over permafrost in summer, or other effect and should avoid any situations or actions which may be hazardous or imprudent.

(3) *Low stress, vibratory loads.*

(a) Design of foundations for radar towers with rotating antennas and structures supporting heavy machinery, turbines, generators, and the like, must consider the response of the foundation and soil mass to the vibration. Evaluation of natural foundation frequency, displacements and settlement may be required. The critical situation may be a condition of resonance which can produce unacceptable displacements and settlements and/or interfere with the operation of the facility. EM 1110-345-3102o gives a design procedure for predicting the resonant frequency and displacement under vibratory loads. Three modes of motion are treated: vertical movement, rotation about a vertical axis, and rocking about a horizontal axis. The equations are based on the elastic half-space concept. Damping is that involved in dissipation of energy with distance; damping as a result of the viscous or internal friction properties of the material is not accounted for. However, as internal damping is small for many soils, especially frozen soils, the procedure is adequate for most cases. A value for the shear modulus of the soil is required. Young's modulus and Poisson's ratio may also be used. The elastic half-space method assumes that the soil mass is more-or-less homogeneous and isotropic. If the soil mass radically departs from the condition, as in the case of a strongly layered soil, a partially frozen condition or similar situation, special design procedures must be employed. Computer codes for calculation are available and two dimensional computer codes are in the state-of-the-art. However, the required properties of the soils are not always directly available from test. This is especially true of frozen soils. Most material property inputs are based on one dimensional plane strain tests. It is seldom possible to exactly reproduce in tests the complex stress and deformation states which govern actual behavior under dynamic loads. Therefore, engineering judgment based on broad experience and knowledge must be employed in choosing test procedures and in analyzing test results to select suitable values for use in computer solutions.

(b) The response of frozen soils to vibratory loads varies with the stress, strain, and frequency imposed by the load, with exterior influences such as temperature and confining pressure, and with the soil characteristics such as void ratio, ice volume/soil volume ratio, degree of ice saturation and soil type. Testing to date has not established definitive relationships with these variables but data indicating general trends are available.

(c) Measurements of velocity of the dilatational wave have been made using seismic methods. Table 4-6 lists velocities for a variety of frozen soils and rocks. If Poisson's ratio is known or assumed, Young's modulus may be calculated from such velocities as follows:

$$E = \frac{{}^{V}c^{2} \quad (1 + \nu)(1\ 2\nu)\ \rho}{1-\nu} \quad \text{(Equation 7)}$$

where: V_c = P-wave velocity
ν = Poisson's ratio
ρ = mass density

***(d)* As the seismic method uses very high rise times, modules values must be considered as upper limits.**

(e) Figure 2-17 shows dynamic moduli and Poisson's ratio determined by Kaplar for various frozen soils at various temperatures". It will be noted that the stiffness decreases drastically as temperature rises and approaches 32 °F. The test procedure used in this case did not allow measurement of internal damping as a material property. No variation of modulus and velocity with frequency or stress level was determined and test frequencies ranged from 830 to 4000 Hz. Stress levels were unknown but were low, such as to give a linear response.

(f) The decrease in stiffness with rising temperature emphasizes the possibility that energy dissipation into the soil may raise the temperature sufficiently to alter the foundation response or even its stability. A pile embedded in frozen soil and depending for its bearing capacity on the adhesive strength between the soil and pile may, under steady-state prolonged vibration, dissipate energy into the soil sufficiently to raise the temperature. The state-of-the-art does not currently allow calculation of energy dissipation into the soil and temperature rise at the soil/pile interface under a given dynamic load. However, the designer should consider the possibility.

(g) Figures 4-56 through 4-60 illustrate the effect of ice volume/soil volume ratio, degree of ice saturation, frequency and stress on the modulus. Complete data including the complex Young's and shear moduli, the corresponding velocity of wave propagation, Poisson's ratio, damping expressed as the tangent of the lag angle between stress and strain, and the attenuation coefficient have been reported by CRREL[176].

Table 4-6. P-wave Velocities in Permafrost (after Barnes''' with some Additions)

Rock Types	Locality and Reference	Seismic Velocity (10^3 ft/sec) Frozen	Seismic Velocity (10^3 ft/sec) Unfrozen	Seismic Velocity (km/sec) Frozen	Seismic Velocity (km/sec) Unfrozen	Est. Ground Temp. °C
Quaternary sediments						
Silt and organic matter	Fairbanks Area, Alaska[a]	5-10	1.8-4	1.5-3.0	0.6-1.2	-1
Alluvial clay	Northway, Alaska[b]	7.8		2.4		-2
Silt and gravel	Fairbanks Area, Alaska[c]	7.7-10		2.3-3.0		-1
Aeolian sand	Tetlin Junction, Alaska[c]	8		2.4		-3
Floodplain alluvium	Fairbanks Area, Alaska[c]	8-14	6.1-7	2.4-4.3	1.9-2.1	-1
Tundra silts, sands, and peats (Gubik Formation, probably saline)	Barrow Area, NPR-4, Alaska (Woolson 1962)	8-8.8		2.4-2.7		-9
(Gubik Formation, probably saline)	Skull Cliff Area, NPR-4, Alaska (Woolson 1962)	7.4-8.9		2.3-2.7		-9
(Gubik Formation, less saline)	Topagoruk Area, NPR-4, Alaska (Woolson 1962)	8-12		2.4-3.7		-9
Gravel	Fairbanks Area, Alaska[a]	13.0-15.2	6-7.5	4.0-4.6	1.8-2.3	-1
Outwash gravel	Tanacross, Alaska[c]	7.6-10		2.3-3.0		-3
Glacier moraine*	Delta Junction, Alaska[c]	7.6-13.2		2.3-4.0		-2
Unclassified sediments	Isachsen, Canada (Hobson 1962)	8.8		2.7		-10
Glacier outwash	Thule, Greenland (Roethlisberger 1961, 1961a)	14.9-15.5		4.5-4.7		-11
Glacier till	Thule, Greenland (Roethlisberger 1961, 1961a)	15.4-16.0		4.7-4.8		-11
Glacier till	McMurdo Sound, Antarctica (Bell 1966)	9.8-13.8	1.6-5	3.0-4.3	0.5-1.5	-20
Loess (dry)	McMurdo Sound, Antarctica (Bell 1966)		1		0.3	-20
Exfoliated granite (dry)	McMurdo Sound, Antarctica (Bell 1966)		4		1.2	-20
Shattered rock (dry)	McMurdo Sound, Antarctica (Bell 1966)		2.6-8		0.8-2.5	-20
Mesozoic sediments						
Mudstone (Ogotoruk Formation)[d]	Ogotoruk Creek, Alaska (Barnes 1960)	14.2	11[d]	4.3	3.4[d]	-5
Mudstone (Ogotoruk Formation)[d, e]	Ogotoruk Creek, Alaska (Barnes 1960)	13.2		4.0		-5
Shale and siltstone (Schrader Bluff Formation)[e]	Fish Creek Test Well 1, NPR-4, Alaska (Chalmers 1949)	8.9-9.8	6.6-7.6	2.7-3.0	2.0-2.3	-8
Shale and sandstone (Chandler Formation)[e, f]	Umiat Test Well 2, NPR-4, Alaska (Legge 1947/48)	12.7		3.9		-7
Shale and sandstone (Nanushuk Group)[e, g]	Simpson and Minga Wells, NPR-4, Alaska (Wiancko 1950)	8.1-8.4	5-7	2.5-2.6	1.5-2.1	-9
Sandstone (Colville Group)	Umiat Area, NPR-4, Alaska (Woolson 1962)	10.7		3.3		-7
Sandstone and shale (Nanushuk and Colville Group)	Meade-Oumalik Area, NPR-4, Alaska (Woolson 1962)	10-14		3.0-4.3		-9
Sandstone (Isachsen Formation)	Isachsen, Canada (Hobson 1962)	11.1		3.4		-10
Paleozoic and older sediments						
Shale (Dundas Formation)	Thule, Greenland (Roethlisberger 1961, 1961a)	12.5-15		3.8-4.6		-11
Sandstone (Narssârssuk Formation)	Thule, Greenland (Roethlisberger 1961, 1961a)	17.0-17.4		5.2-5.3		-11
Quartzite (Wolstenholme Formation)	Thule, Greenland (Roethlisberger 1961, 1961a)	18.4-19		5.6-5.8		-11
Dolomite (Narssârssuk Formation)	Thule, Greenland (Roethlisberger 1961, 1961a)	18.9-19.3		5.8-5.9		-11
Metamorphic rocks						
Schist (Birch Creek Schist)	Fairbanks Area, Alaska[a]	13-16		4.0-4.9		-1
Gneiss	Thule, Greenland (Roethlisberger 1961, 1961a)	20-20.8		6.0-6.3		-11

[a] Obtained from H.G. Taylor, 1938, Report on geophysical work by the seismic method in placer deposits of Fairbanks District of Alaska, unpublished report to U.S. Smelting, Refining & Mining Co.
[b] Unpublished data from J.H. Swartz and E.R. Shephard, U.S. Geol. Survey, 1946
[c] Data by author in 1952
[d] The Materials Testing Laboratory, U.S. Army Engineer District, Alaska, tested cores from this well and found that 5 porosity measurements averaged 6.4% and that dynamic measurements on unfrozen cores gave elastic moduli which may be used to compute a velocity in the unfrozen rocks of about 11,000 fps
[e] Measurements by velocity logs of wells; rest measured by refraction
[f] Porosity measurements of 44 cores from Umiat Test Well #2 averaged 13.5% (Collins 1958)
[g] Porosity measurements of 15 cores from Simpson Test Well #1 averaged 30.8% (Robinson 1959)

(Courtesy of Building Research Advisory Board, NAS-NRC)

For these data, the test technique employed was that given by Stevens[93]. Complex moduli may be used as equivalent to elastic moduli when damping is normally small.

(h) As shown in figure 4-56, complex Young's modulus of 100 percent ice-saturated, non-plastic frozen soils increases sharply as the ice volume/soil volume ratio decreases. As the ratio increases the modulus approaches that of ice. The relationship shown does not appear to apply for plastic soils.

(i) Figures 4-57 and 4-58 show the relationships of complex shear and Young's moduli to percent ice saturation. The moduli decrease rapidly with decrease in percent saturation, obviously approaching the moduli in a non-frozen state.

(j) Figures 4-59 and 4-60 show the relationship between shear modulus, stress level, and frequency. Within the test ranges the relative effects are small except for partially saturated clay. However, the effect of stress and/or frequency on the shear modulus could be significant for stresses and/or frequencies outside these test ranges. Therefore, these relationships are sufficiently important to warrant consideration when choosing a modulus for design.

(k) Smith[91] has published data on properties of snow and ice under vibrating loads.

(l) The data presented in this manual and in the reference publications serve only as a guide and it will be necessary in most problem cases to carry out a test program covering the range of soil conditions, test freq

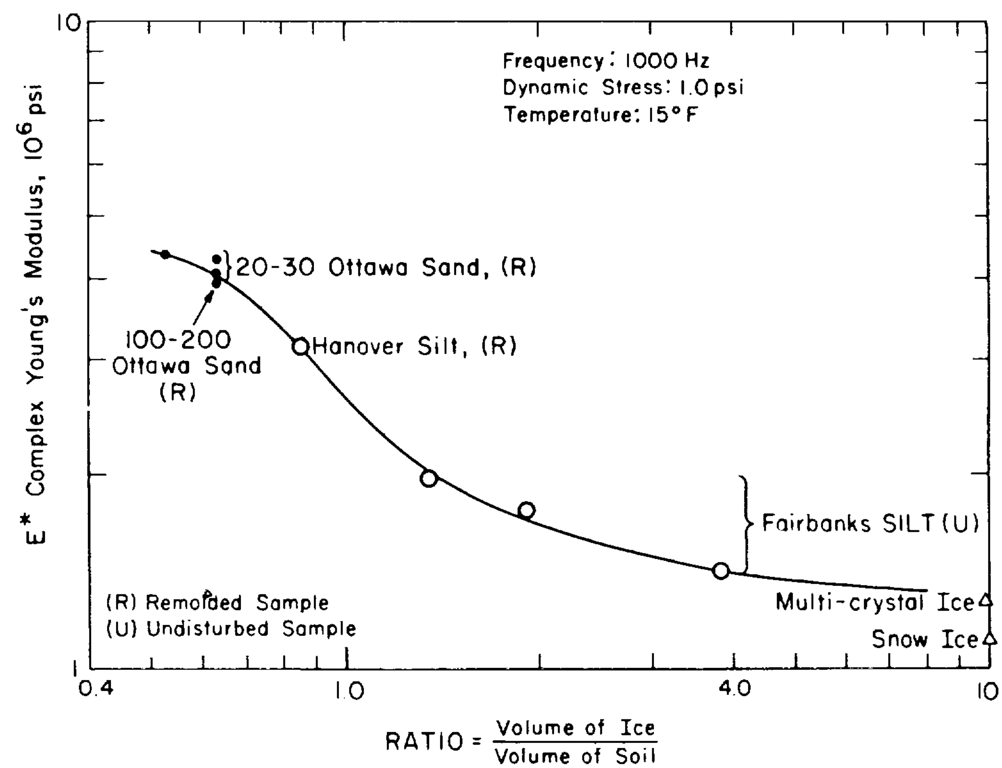

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Figure 4-56. Complex dynamic Young's modulus vs. volume ice/volume soil ratio for frozen saturated, non-plastic soils.

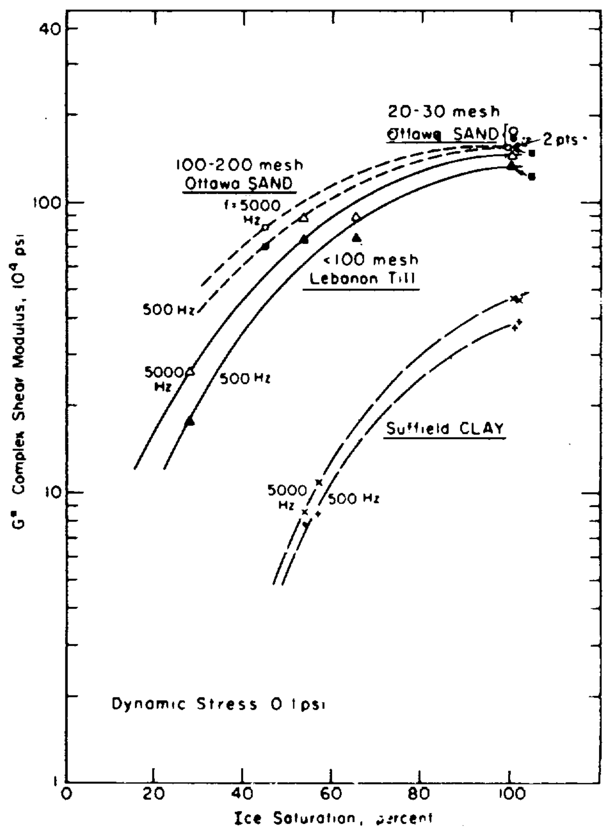

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Figure 4-57. Complex dynamic shear modulus vs. ice saturation.

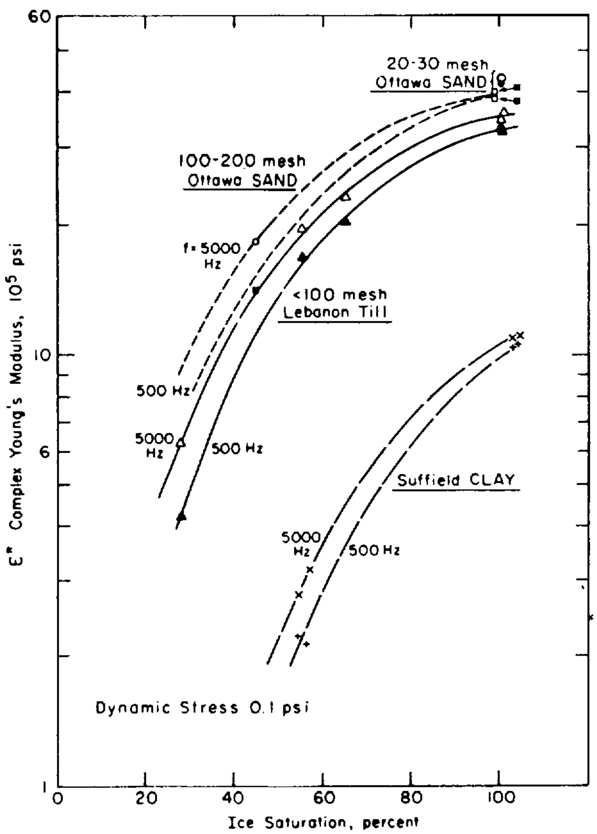

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Figure 4-58. Complex dynamic Young's modulus vs. ice saturation.

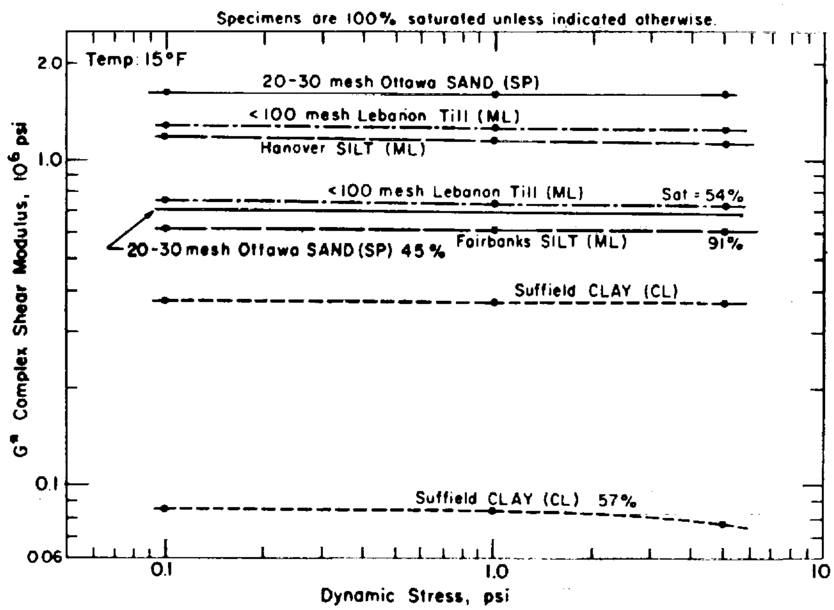

Figure 4-59. Complex dynamic shear modulus vs. dynamic stress at constant frequency (500 Hz), temperature 15 °F (by CRREL).

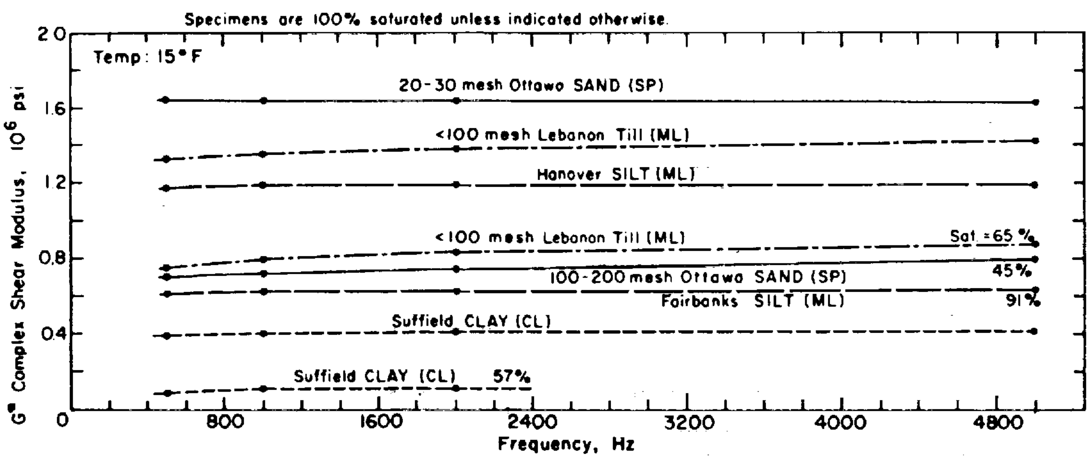

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Figure 4-60. Complex dynamic shear modulus vs. frequency at constant dynamic stress (1.0 psi), temperature 15°F (by CRREL).

nuencies, stress levels, and temperatures, applicable for the specific site.

4-7. Design of footings, rafts and piers.

a. General. Footings, rafts and piers are considered together in this section because they share the common characteristic of developing load supporting capacity through the bearing of a horizontal surface of adequate dimensions on the supporting soil stratum.

(1) Suitable foundations of these types should:

(a) be safe with respect to bearing capacity failure

(b) keep displacement, settlement, heaving, or creep deformations, and, when required, resonant frequency within acceptable limits.

(c) be economical to construct and maintain.

(2) Foundation design must satisfy all three of these criteria. However, in some cases one or two of the criteria may have predominant importance.

(3) A *footing* is an enlargement of a column or wall to distribute concentrated loads over sufficient area so that allowable pressure on the soil will not be exceeded. A spread footing, individual column footing, or isolated footing is usually an individual square or circular footing placed beneath a column or post. A combined footing is a footing carrying more than one column or post. A footing that supports a wall is a continuous or wall footing. A shallow footing is a footing whose width is equal to or greater than the vertical distance between the surface of the ground and the base of the footing. A sill or mud sill is a structural piece such as a timber which rests directly on the soil and supports the structure.

(4) With proper design care all these types can be used successfully for arctic and subarctic structure foundations under the particular conditions to which they are suited, although they may not always be as economical as alternatives such as piles. Individual column footings are preferred to continuous footings because of the greater risk of structural foundation damage for the continuous type of footing in frost areas. Examples of all these footing types are shown in figures 4-13 through 4-17, 4-22 through 4-24 and 4-27. As illustrated by these figures, footings on frost-susceptible permafrost soils may be placed at the surface of foundation gravel mats as in figures 4-13 through 4-15, and may be placed within the gravel mat as in figure 4-24, or may be placed below the depth of maximum seasonal thaw which will exist after construction, as illustrated in figures 4-16, 4-17 or 4-23. In the latter case a granular layer of clean gravel or sand should be used immediately below the footing to provide a suitable working and placement surface; it may also serve to reduce bearing pressures on the underlying materials. During construction in warm weather this granular layer also provides some protection of underlying thaw susceptible permafrost during the footing installation.

(5) For frost-susceptible soils in seasonal frost areas footings should normally be supported below the depth of seasonal frost penetration, whether the structure is heated or not. While it is true that such footing depth may appear unnecessary for many types of heated structures in seasonal frost areas because of the protective effect of heat losses into the ground, it is unrealistic to assume that a facility will continue to be heated if it is placed on standby status at some future date. Placement of a non-frost-susceptible fill of sufficient depth on the site would avoid this requirement.

(6) If foundation soils are clean, granular and nonfrost-susceptible, conventional temperate zone foundation practice may be used in both seasonal frost and permafrost areas and footings should be placed at a minimum depth of 4 feet. This depth is recommended in order to minimize seasonal thermal and mechanical effects, such as seasonal expansion and contraction, which are most intense in the upper layers of the ground and tend to impose stresses on the structure.

(7) A variation of the continuous footing is the sill or mud sill type foundation, usually used for small structures having light floor loads or for temporary buildings. If only small-dimension sill members are used, without ventilation, this option is limited to nonthaw-susceptible granular foundation materials or under less favorable foundation conditions to very temporary use beneath small structures with the aid of a granular mat to distribute differential movements and jacking and shimming provisions to allow adjustment of differential movements. However, the latter usage is nor recommended, even for temporary construction facilities, unless heated. If large dimension sills are used, marginal ventilated foundation designs are possible for small structures, as by using through beams across the minimum dimension of the structure.

(8) *A raft or mat foundation* is a combined footing which covers the entire area beneath the structure and supports the walls and structural columns. It is usually employed where heavy floor loads are required, such as in hangars, garages and warehouses. It acts as a unit and minimizes differential movements which could occur in the use of individual footings. As employed in permafrost areas, ordinary raft foundations are reinforced concrete slabs on gravel mats, as illustrated in figures 4-25 and 4-26, employing various means of insulation and ventilation or refrigeration between the floor and the underlying gravel mat (paras 4-2c and 4-2d). Care must be taken to insure that all parts of the foundation are protected by insulation and cooling provisions, including areas beneath the walls.

(9) In an intermediate type of foundation, a slab is used to support heavy floor loads, but the support

Strip footing: $q_u = \frac{Q}{B} = cN_c + \gamma DN_q + 0.5\gamma BN_\gamma$

Rectangular footing: $q_u = \frac{Q}{BL} = cN_c \left(1 + 0.3\frac{B}{L}\right) + \gamma DN_q + 0.4\gamma BN_\gamma$

Circular footing: $q_u = \frac{Q}{\pi R^2} = 1.3cN_c + \gamma DN_q\ 0.6\gamma RN_\gamma$

where: Q = total load bearing capacity of footing, lb.

q_u = ultimate bearing capacity, psf

c = allowable long-term cohesion (see paragraph 4-4)

γ = unit weight of soil

L = length of footing

N_c, N_q, N_γ = dimensionless bearing capacity factors

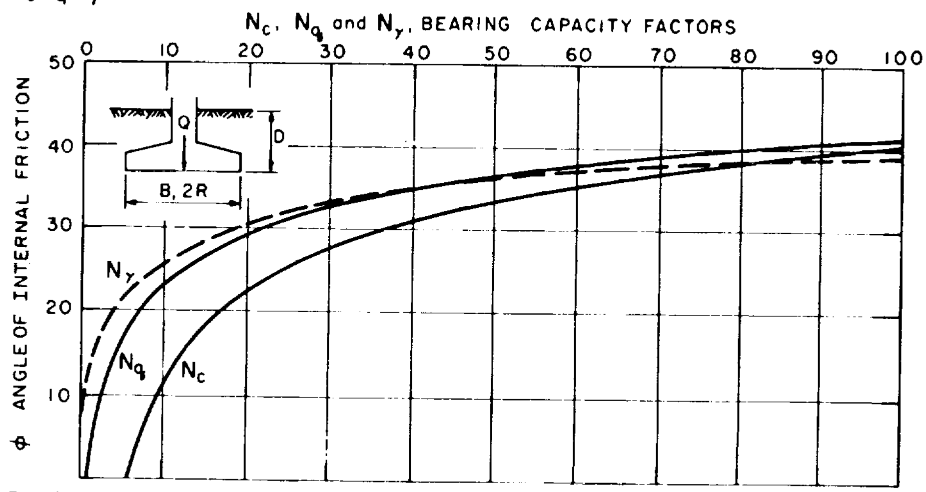

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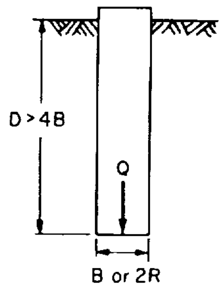
Figure 4-61a. Bearing capacity formulas[5]. (Ultimate bearing capacity of shallow foundations under vertical centric loads.)



Strip loading: $\frac{Q}{L} = cBN_c + \gamma DB + 2Df_s$

Square loading: $Q = cB^2N_c + \gamma DB^2 + 4BDf_s$

Circular loading: $Q = c\pi R^2N_c + \gamma D\pi R^2 + 2\pi RDf_s$

Notes:

1. Bearing capacity factors N_c for foundations in $\phi = 0°$ material:

B/L	N_c
1 (square or circle)	9.0
0.5	8.2
0 (strip footing)	7.5

2. Bearing capacity factors based on a smooth base (base shear stress = 0) and D/B greater than 4.
3. The skin friction f_5 should not exceed the-minimum sustained tangential adfreeze bond strengths recommended in paragraph 4-8f(1).

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Figure 4-61b. Bearing capacity formulas[1]. (Ultimate bearing capacity of deep foundations in c-∅ material.)

system for wall and column loads is separate, as illustrated in figures 4-24, 4-27 and 4-28. Choice may depend on either structural or economic factors, or both.

(10) A pier is a prismatic or cylindrical column that serves, like a pile or pile cluster, to transfer load to a suitable bearing stratum at depth, as illustrated in figure 4-61b. It may be noted in figure 4-61b that the bearing capacity increases as the square of the radius of the pier in two of the three factors making up the total bearing for the circular pier, thus suggesting rapid increase of bearing capacity with diameter of pier.

(11) However, it should also be noted that the ultimate frost heaving force which can be developed on a pier or on the stem of a footing is a function of the surface area of the member in contact with the seasonally frozen ground and hence of the diameter of a cylindrical member. The frost heaving force is capable of fracturing members which are weak in tension and which are solidly anchored below the seasonal frost zone. Footing or pier members in the seasonal frost zone which may be placed in tension by frost heave forces should be made strong enough to resist such tensile stresses. The amount of steel reinforcement in concrete members should be sufficient to prevent cracking of concrete and the consequent exposure of steel to moisture. To prevent foundation uplift, bases of footings should be large enough to resist frost heave uplift through passive soil reaction of the base projections against overlying materials, or pier-type foundations should extend to sufficient depth to resist such uplift through skin friction or adfreeze bond on lateral surfaces. Uplift resistance contributed by dead load from structure and weight of foundation should be taken into account. When practical, surfaces in contact with the frozen soil may be battered so that heave of the frozen layer will tend to reduce the contact and thereby minimize heaving forces. It is also desirable that vertical foundation members extending upward through the annual frost zone from underlying pads be as small in cross section as possible to minimize the total heave seasonal frost zone during thaw, care must be taken to avoid column instability in thinned-down members. Frost heave forces may be estimated as described in paragraph 4-31.

force. Of course, if foundation members are isolated from the annual frost zone by methods as described in paragraph 4-31 such requirements are reduced. Since negligible lateral support may be provided in the

(12) Innumerable variations and combinations of structural designs of foundation members are possible. Piers may be stepped, tapered or made hollow, footings and columns may be combined in the form of pedestals, etc. It is not possible to anticipate and give guidance for all such variations. However, the principles outlined herein may be combined as needed to fit any situation.

(13) Allowable bearing values for shallow footings and sills supported on well-drained granular mats may be based on unfrozen strengths of these materials. The thickness of granular material between footing and underlying natural soil must, however, be sufficient to reduce concentrated stresses on the natural soil to tolerable levels.

(14) Allowable bearing values for footings, rafts and piers supported on frozen materials should be determined using analytical procedures and factors of safety outlined in TM 5-818-1/AFM 88-3, Chapter 7 [5] (F.S. = 2.0 for dead load plus normal live load and 1.5 for dead load plus maximum live load) and the design strength values determined as described in paragraph 4-4 for ultimate strength analysis, assuming that control against degradation of permafrost has been carefully provided in the design. Since most footings supported on permafrost are placed near the top of permafrost where ground temperature may rise fairly close to 32 °F in summer and fall, soil design values within 1° to 3 °F of 32 °F are usually applicable. Stresses in the weakest underlying strata must be kept within tolerable levels. Creep deformations estimated in accordance with paragraph 4-5 may also exert overriding control over allowable bearing values.

(15) Those of the analytical bearing capacity analyses presented in TM 5-818-1/AFM 88-3, Chapter 7[5] which are most typically applicable for frozen materials are illustrated in figure 4-61, covering shallow and deep foundations.

(16) For computation of bearing capacity of unfrozen soils and for detailed general guidance, reference should be made to applicable procedures outlined in TM 5-818-1/AFM 88-3, Chapter 7 [5].

(17) Design bearing pressures of 2500, 3000 and 4000 psf on frozen soils have been used satisfactorily in the designs shown in figures 4-15, 4-16 and 4-27, respectively. A bearing pressure of 12,000 psf has been used satisfactorily on very bouldery till with estimated highest soil temperature at the base of the footing of 25 °F, near Thule AB, Greenland[86].

(18) Settlement of footings on permafrost from pressure melting or extrusion of ice from soil voids should be assumed negligible under normal footing loadings. A footing bearing value of 3600 psf, for example, corresponds to 25 psi. In terms of melting point lowering this corresponds to only about 1/40°F. As indicated in figures 2-12, 2-15 and 4-53, the potential strength and deformation properties of ice are such that strata of hard, high-density ice in the foundation may exhibit better bearing characteristics than may frozen soils, especially at temperatures above about 25°F. However, porous ice may be compressible.

(19) Design procedure for predicting the resonant frequency and displacement under vibratory loads is given in EM 1110-345-310[20].

(20) Design guidance with respect to other aspects of footing, raft and pier design are presented in other paragraphs of this manual as follows:

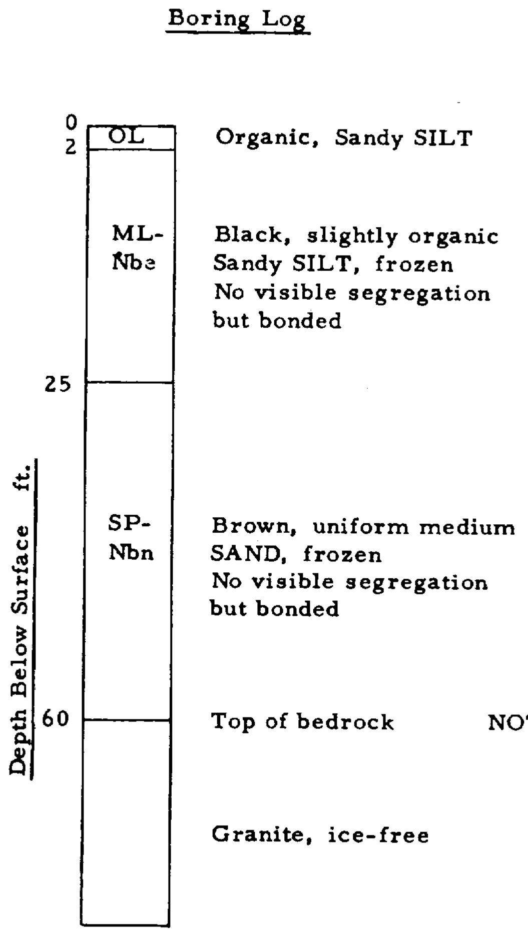
Settlement analysis, paragraph 3-4f
Thermal stability analysis and control, paragraph 4-2
Control of movement and distortion from freeze and thaw, paragraph 4-3
Estimation of creep deformation, paragraph 4-5

b. Illustrative examples for the design of footings in permafrost.

(1) Steps required for design of footings on permafrost. Determine required depth of footing (a above); determine the temperature distribution in the permafrost with respect to the base of the footing, for the critical period of the year; check the bearing capacity of a trial footing size in this critical period of the year using standard soil mechanics theory (a above) with properties of the frozen soil determined by field or laboratory tests (see chap 3); adjust the footing dimensions to obtain the desired factor of safety; and make a settlement analysis. Determine the distribution of the imposed vertical stress with depth beneath the footing. Compute the deformation of a free standing column of frozen soil, using creep equations and constants obtained from laboratory unconfined compression creep tests (para 4-5). If the estimated settlement is excessive, enlarge the footing or otherwise revise the design to reduce the estimated settlement to a tolerable amount.

(2) Example of design of an isolated square footing.

(a) Assumptions:
Permanent structure, 25-year life
Soil conditions as shown in boring log (fig. 4-62)
Properties of the 2-feet-thick organic sandy silt layer (fig. 4-62) essentially the same as for the underlying sandy silt
Footing design of general type shown in figure 4-63
Column load 150 tons
Required factor of safety with respect to ultimate bearing capacity = 2.0
Air space provided between building floor



Soils Data

Sandy SILT:

Dry unit weight,
γ_d = 85 lb/ft^3

γ = 115 lb/ft^3

Water content,
w = 33%, based on weight of solids

Thermal conductivity,
K = 1.2 Btu/ft.hr. °F* (frozen) and
0.68 " " " " (unfrozen)

SAND:

γ_d = 105.0 lb/ft^3

w = 20%

K = 2.0 Btu/ft.hr. °F* (frozen)

NOTE: * Values for K may be determined by test but are usually taken from charts in TM 5-852-6 [14]

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Figure 4-62. Boring log and soils condition (by CRREL).

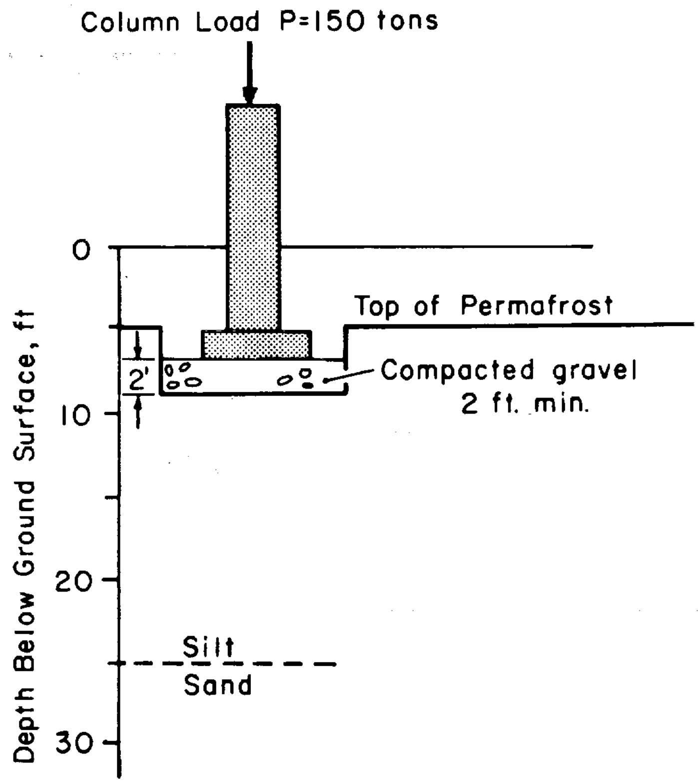

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Figure 4-63. Case of isolated square footing (by CRREL)

and ground surface
Mean annual air temperature (MAT) = 23°F.
Ground temperature below zone of seasonal variations (at 50 feet, measured) = 27 °F.
Design air thawing index, I = 2900
Length of thawing season, t = 150 days
Conversion factor for air thawing index to surface thawing index, n = 1.0 for building-shaded area (see table 4-1 and TM 5-852-6), dimensionless. (Note that this factor does not apply for the freezing period or for the overall annual heat exchange.)

(b) Determining required depth of footing. The footing should be founded below the top of permafrost for stable bearing. To establish this position the depth of seasonal thaw must be determined. If previous measurements of depth of thaw are available, these should be used, adjusted if necessary to correspond with the design air thawing index condition. Otherwise the thaw depth should be determined by computation using procedures and terminology of TM 5-852-6/AFM 88-19, Chapter 6'".

As indicated in TM 5-852-6YAFM 88-19, Chapter 6 use the equation,

$$X = \frac{\lambda\sqrt{48KnI}}{L}$$

where

X = depth of thaw, ft
L = volumetric latent heat of fusion, Btu/ft^3
λ = a coefficient which takes into consideration the effect of temperature changes in the soil mass (other factors are as previously defined)
λ = is determined from TM 5-852-6/AFM 88-19, Chapter 6, figure 13, using values of α and μ calculated as follows:

$$\text{Thermal ration } \alpha = \left|\frac{v_o}{v_s}\right|$$

Average thaw-season surface temperature differential

$v_s = \frac{nI}{t} = \frac{(1.0)\ (2900)}{150} = 19.3$ (above 32°F)

Initial temperature differential

v_o = MAT -32 = 23-32 = 9°F (below 32°F)

$\alpha = \frac{9}{19.3} = 0.47$

Fusion parameter $\mu = v_s$

Average volumetric heat capacity, C = λd (c + 0.75 w/100)

For silt: Dry unit weight, λd = 85 lb/ft^3.

Moisture content of soil (percent of dry weight), w = 33 percent

Specific heat of dry soil, c = 0.17. (Average value for near 32°F; TM 5-852-6/AFM 88-19, Chap 6[14])

C = 85[0.17 + 0.75 (33/100] = 35.5 Btu/ft^3

L = 144 (λd) (w/100) = 144 (85) (33/100) = 4050 Btu/ft^3

λ = 0.88 (from TM 5-852-6/AFM 88-19, Chap 6, fig. 13)

Since the annual thaw zone includes both frozen and unfrozen soil except at the start and the end of the thawing season, an average value of thermal conductivity, K, is the best approximation for this condition. Select individual K values from figures 3 and 4 of TM 5-852-6 (they may also be determined by test). These have been shown in figure 4-62. Then,

$K_{ave} = \frac{1}{2}[K_{unfroz} + K_{froz}] = \frac{1}{2}[0.68 + 1.2] = 0.94$ Btu/ft hr

Estimated depth of thaw $X = \frac{\lambda\sqrt{48knI}}{L} = \frac{0.88\sqrt{48(0.94)(1.0)(2900)}}{4050}$

The footing should be founded a foot or more below the top of the permafrost, depending on the reliability of the data used in the estimate and the degree of confidence that the assumed thermal regime will be maintained. In this case, a depth of 7 feet is used with a footing design of the general type shown in figure 4-63. Because stresses are most intense within about one to one and one-half diameters below the base of the footing, as shown in figure 4-64, temperatures within this depth are most critical. By placing high-bearing value material within the most critical part of this depth, as illustrated by the gravel in figure 4-63, design certainty can be increased.

At the perimeter of the building where transition occurs from the shaded, cooler interior surface under the building to the unshaded natural ground surface, the building should cantilever out beyond the footings or special shading should be provided for sufficient distance to ensure maintenance of design ground temperature conditions under the footings. Ground temperatures under individual footings should be as nearly the same as possible in order to obtain uniform support. By successfully achieving an n = 1.0 condition for the actual foundation support area, for the thaw season, the permafrost table may be expected to become somewhat higher under the building than under adjacent non-shaded areas.

(c) Determining temperature distribution with depth below base of footing for critical period of year. Given that the highest temperature at the top of permafrost is 32°F and the permafrost temperature at a depth below the influence of annual temperature fluctuations is 27°F it is assumed that erection and operation of the structure does not significantly affect the mean annual temperature at the latter depth. Studies of field data show that the temperature of permafrost, T_{x5} at depth X below the permafrost table may be determined from the expression:

$T_X = 32 - (A_O - A_X)$

where:

Ao = amplitude of temperature wave that the top of permafrost above the temperature at the depth of no annual variation.

In this case,

$A_o = 32 + 27 = 5$°F

and from p. 36, TM 5-852-6/AFM 88-19, Chapter 6

$A_x = A_o \exp(-X\sqrt{\pi/ap})$

where:

a = thermal diffusivity = K/C

P = period of sine wave, 365 days

The footing size is in this case assumed to be small enough so that the foundation temperatures are not significantly affected by the differing thermal properties of the footing and underlying gravel.

For frozen silt:

K = 1.2 Btu/ft hr °F

$C = \gamma\ (c + 0.5\ w/100) = 85\ (0.17 + 0.5\ (33/100))$

$= 28.5$ Btu/ft^3

$a_{silt} = K/C = \frac{1.2}{28.5} = 0.042\ ft^2/hr = 1.01\ ft^2/day$

$A_x = 5 \exp(-X\sqrt{\pi/(1.01)(365)}) = 5e^{-0.0923X}$

and

$T_x = 32 - (5-5e^{-0.0923X}) = 32 - 5 + 5e^{-0.0923X}$

$= 27 + 5e^{-0.0923X}$

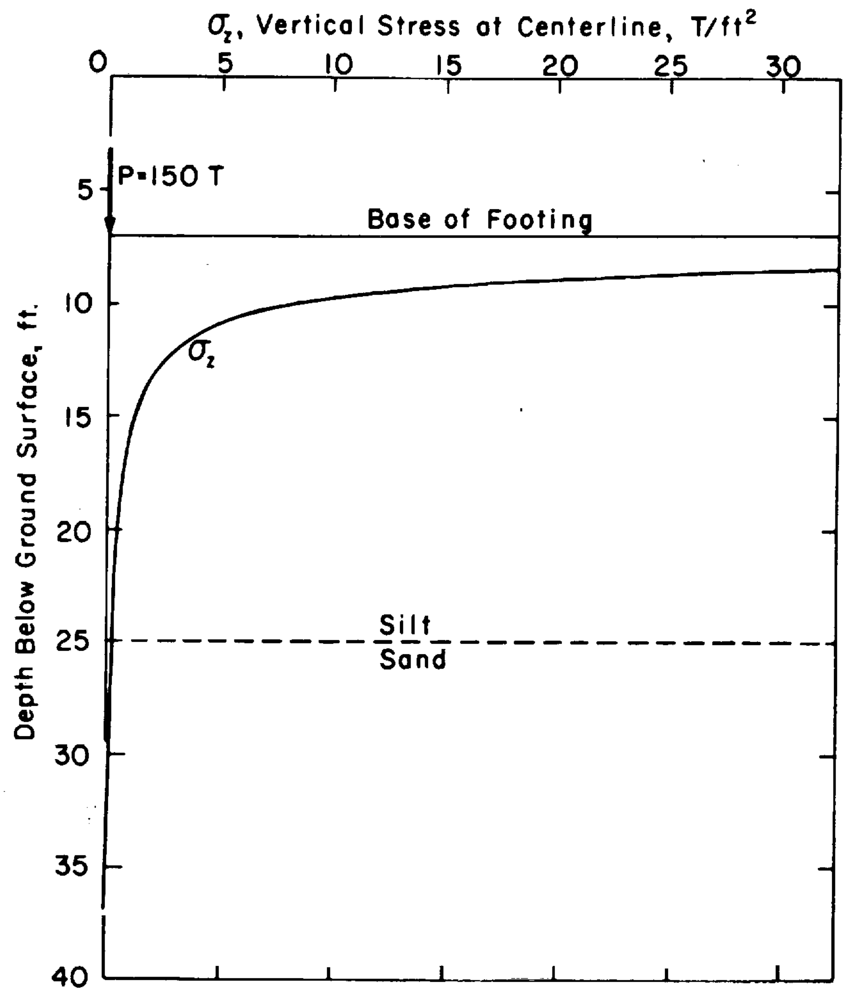

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Figure 4-64. Vertical stress at centerline (by CRREL).

The equation for Tx predicts the maximum temperatures occurring at particular depths. These maximum temperatures do not occur simultaneously but the assumption that they do is conservative for this situation.

The diffusivity of the sand is greater than that of the silt, which will induce higher temperatures in the sand layer than would result if all the soil were silt. An adjustment can be made for the layered soil condition by using the procedure outlined below. To determine temperature distribution with depth in the sand, it is necessary to convert the sand layer to an equivalent silt layer. From page 38, TM 5-852-6/AFM 88-19, Chapter 6[14], the thicknesses are proportional to the square roots of the thermal diffusivities.

Diffusivity for silt,

$$a_{silt} = 1.01\ ft^2/day$$

Diffusivity for sand,

$$C_{sand} = \gamma_d\ [c + 0.5\ (w/100)] = 105.$$

$$[0.17 + 0.5\ \frac{(20)}{(100)}]$$

$$C_{sand} = 28.3\ Btu/ft^3\ {}^\circ F$$

$$K_{sand} = 2.0\ Btu/ft^3\ hr\ {}^\circ F\ (frozen)$$

$$a_{sand} = K/C = \frac{2.0}{28.3} = 0.0706\ ft^2/hr = 1.7\ ft^2/day$$

$$\text{Ratio} = \frac{\sqrt{a_{silt}}}{\sqrt{a_{sand}}} = \sqrt{\frac{1.01}{1.7}} = 0.77$$

i.e. I foot of sand is equivalent to 0.77 foot of silt, as regards temperature penetration under transient conditions.

Values of T_x are computed for the entire depth of possible interest assuming silt, and depth adjustments are then applied in the sand layers. Computations for selected depths are shown in table 4-7 and the temperature distribution with depth is illustrated in figure 4-65. For use in settlement computations, a simplified temperature distribution is also shown in the figure.

(d) Checking bearing capacity in critical period of year. Use equation:

$$q_u = 1.3c\ N_c + \gamma D\ N_q + 0.4\gamma B\ N_\gamma$$

from figure 4-61, using terminology thereon, or from TM 5-818-1/AFM 88-3, Chapter 7[5], for square footings.

Neglecting internal friction ($\phi = 0$ and $N_\gamma = 0$)

$$q_u = 1.3c\ N_c + \gamma DN_q$$

From figures 4-61 and 4-62.

$$N_c = 5.7$$
$$N_q = 1.0$$
$$\gamma = 115\ lb/ft^3$$
$$D = 7\ ft\ \text{(at base of footing)}$$

The cohesion, c, of the frozen silt must be determined from creep test data at about 30°F (average temperature in the top 2 feet of silt as shown in fig 4-65.) Take failure cohesion at 25 years.

Unconfined compressive strength determined by conventional laboratory test is

$$q_w = 450\ psi = 32.4\ T/ft^2$$

Results of laboratory unconfined compression creep tests are:

Percent of q_w	Applied Creep Stress	Time of Failure
60	270 psi (19.4 T/ft²)	$t_f = 0.027$ hr
40	180 psi (13.0 T/ft²)	$t_f = 0.24$ hr

Using equation 3 (para 4-4):

$$\sigma_{ult} = \frac{\beta}{\ln(t_f/B)} = \frac{\beta}{\ln t_f - \ln B}$$

$$\ln B = \ln t_f - \frac{\beta}{\sigma_{ult}}$$

Substituting:

$$\ln B = \ln 0.027 - \frac{\beta}{270}$$

and

$$\ln B = \ln 0.24 - \frac{\beta}{180}$$

Solving for B and β and substituting back,

$$\sigma_{ult} = \frac{1180}{\ln t_f = 6.67}$$

For failure time of 25 years $= 2 \times 10^5$ hr:

$$\sigma_{ult} = \frac{1180}{\ln 2x10^5 + 6.67} = 62.2\ psi$$

$$\text{Cohesion}\ \sigma_{ult} = \frac{62.2}{2} = 31.1\ psi = 2.2\ T/ft^2.$$

Since the gravel layer beneath the footing is a high bearing value material, using D = 7 feet for depth of footing might be too conservative. However, using D = 9 feet at the base of the gravel layer might not be conservative enough. Therefore, both values will be checked below to determine the effect on indicated capacity. For D = 7 feet, Ultimate Bearing Capacity, qu = 1.3 c Nc + yDNq

$$q_u = 1.3\ (2.2)\ (5.7) + \frac{(115)(7)(1.0)}{2000} = 16.3 + 0.4$$

$$q_u = 16.7\ T/ft^2$$

Table 4-7. Computation of Temperature Below Top of Permafrost.

Depth Below Top of Permafrost		$0.0923X$	$e^{-0.0923X}$	$5e^{-0.0923X}$	Temperature, T_X $=(27 + 5e^{-0.0923X})$ (^{o}F)
In Silt, X (ft)	In Sand, X' (ft)				
0	-	0.0	1.0	5.0	32.0
1	-	0.0923	0.912	4.56	31.6
2	-	0.184	0.832	4.16	31.2 Base of footing
3	-	0.277	0.76	3.80	30.8
4	-	0.369	0.69	3.45	30.4 Bottom of gravel layer
5	-	0.461	0.63	3.15	30.1 Silt
6	-	0.554	0.58	2.90	29.9 "
7	-	0.646	0.52	2.60	29.6 "
10	-	0.923	0.39	1.95	28.9 "
12	-	1.11	0.33	1.65	28.6 "
15	-	1.38	0.25	1.26	28.2 "
20	-	1.85	0.157	0.78	27.8 Bottom of silt layer
(25)	26.5	2.31	0.099	0.49	27.4 Sand
(30)	33.0	2.76	0.062	0.31	27.3 "
(35)	39.5	3.23	0.039	0.20	27.2 "
(40)	46.0	3.69	0.025	0.12	27.1 "
(50)	59.0	4.61	0.010	0.05	27.0 "

NOTE: The temperature distribution in the silt layer is the same as if all the soil were silt and the temperature distribution in the sand is obtained by first computing the temperature as if the layer were silt and then adjusting the depths beneath the silt-sand interface, In this case:

$X' = \frac{1}{0.77}(X-20) + 20$ = Actual distance from top of permafrost to point in sand

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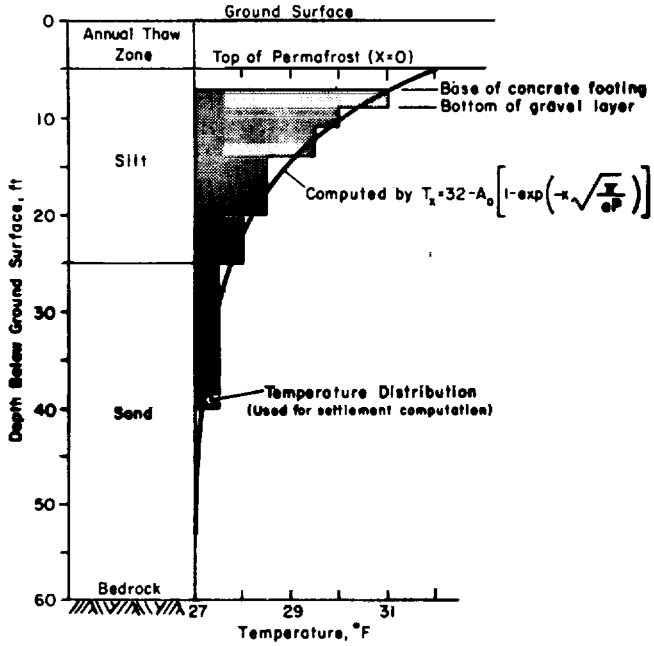

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Figure 4-65. Temperature distribution for permafrost below footing. Temperature distribution should be checked by measurement (e.g. by thermocouples if possible.)

For D = 9 feet, $q_u = 1.3\ (2.2)\ (5.7) + \frac{(115)(9)(1.0)}{2000} = 16.3 + 0.5$

$q_u = 16.8\ T/ft^2$

The difference is not significant and the lower of the two,

q_u = 16.7 T/ft², will be used.

Total Bearing Capacity = q_u × area of footing

Using 4 feet × 4 feet footing:

Capacity = 16.7 × (4×4) = 267 T.

$$\text{Factor of Safety FS} = \frac{\text{Total Bearing Capacity}}{\text{Design Footing Load}} = \frac{267}{150}$$

$$= 1.78\ (< 2).$$

Therefore use 4-1/2 ft × 4-1/2 ft footing which gives:

Total Bearing Capacity = 16.7 × (4-1/2 × 4-1/2) = 338 T.

Factor of Safety = $\frac{338}{150}$ = 2.25 (> 2) OK.

(e) Making settlement estimate. Consider the isolated footing as a point load near the surface of a semi-infinite solid (conservative assumption because of depth and increased bearing area).

1. Stress distribution. For simplicity, use Boussinseq's equation for point load. (See Terzaghi and Peck [198], 2nd Ed., p. 271.)

Vertical stress

$$\sigma_z = \frac{3P}{2\pi}\frac{1}{z^2} = \frac{3(150)}{2\pi}\frac{1}{z^2} = \frac{71.7}{z^2}$$

where z = distance below base of footing.

The computed stress distribution is shown in figure 4-64.

2. Creep settlement computation.

Assume:

Load P is distributed uniformly over the end of a soil column with cross section equal to the base area of the footing with stress decreasing progressively in the column to the depth where the stress is negligible, as indicated in figure 4-66.

Vertical movement is the result of unconfined compression creep of the frozen column of soil directly beneath the footing. (This assumption is on the safe side since creep-reducing effects of lateral confinement are neglected.)

Total creep movement is the sum of the creep of all the zones of soil in the soil column.

Creep in the compacted gravel is neglected.

Temperature distribution is as shown in figure 4-65 and as computed in table 4-7. (The approximate distribution assumed for computation is also shown in fig 4-66).

The amount of creep deformation can be estimated by the following methods:

By using constants from table 4-5 and equation 4 from paragraph 4-5.

By performing unconfined compression creep tests on undistributed samples of the foundation soil and using equation 6 of paragraph 4-5.

The first method will give a rough estimate. An example of computations by this method using values from table 4-5 for a silt similar to the soil under the footing, at about the same water content, is shown in table 4-8. Use of a value of 25 years of time, t, in these computations is a simplification and quite conservative in that it assumes that ground temperatures remain throughout the year at the same level as during the "critical period." Since ground temperatures are somewhat colder during a considerable portion of the year, it is clear that the length of time required to attain the settlements computed in table 4-8 (and in table 4-10 as well) is somewhat longer than the 25 years. Computing the settlement using the coldest temperatures to be anticipated during the year (24°F for Zone A, etc.), with all other factors the same, results in a 25-yr settlement of 0.2 in., 1/5 that determined in table 4-8.

The second method gives a more accurate prediction for the specific case. The unconfined compression creep tests should be performed at the design stress level and at the predicted temperature of the foundation soil. A plot of unconfined compression creep test results for a silt at 29.5°F under applied stress of 50 psi (3.6 T/ft²) and a sample computation are shown in figure 4-67 (for Zone B).

Using equation 6 and the data from the test, the relationship between strain and time becomes, for Zone B:

$$\textbf{Strain, } \epsilon = 4.55 \times 10^{-4}\ [t^{0.099} - 1] + 0.00051$$

A similar relationship must be obtained by tests performed on soil from each zone in the "soil column" beneath the footing for the critical temperature and the stress conditions that exist

The sum of the deformations from all the zones for a given time will constitute the estimate of the total settlement.

For demonstration purposes assume time, t = 25 yr ≈ 2 × 10⁵ hr.

Zone A, Strain, ϵ = 4.55 × 10^{-5} + 6.68 × 10^{-5} = 8.84 × 10^{-5}

Creep test not performed on Zone C at required temperature; strain data interpolated from a general formula and average values to complete example.

Deformation:
Zone A, 0.0313 x 2 ft = 0.0626 ft
Zone B, 0.00158 x 3 ft = 0.0047 ft
Zone C, 0.0000884 x 6 ft = 0.0005 ft
TOTAL = 0.0678 ft = 0.81
Say 1 in., at end of 25 years.

The details for the field test of the third method are given in paragraph 4-5. Actual data for a field test under the conditions assumed in this example are not available. As indicated in paragraph 4-5, the various requirements for the field test make this approach difficult to employ and the designer will generally have to rely on laboratory unconfined compression creep tests in estimating creep deformation.

(3) Example of design of a uniformly loaded square raft foundation.

Assumptions: assume the same soil conditions as shown in figure 4-62 and a foundation of the general type shown in figure 4-68.

Determining required depth of base of raft. The analysis is the same as for the isolated square footing in the preceding example.

Determining the temperature distribution with depth below foundation for critical period of year. The analysis is the same as for the isolated square footing in the preceding example.

Checking bearing capacity in critical period of year.

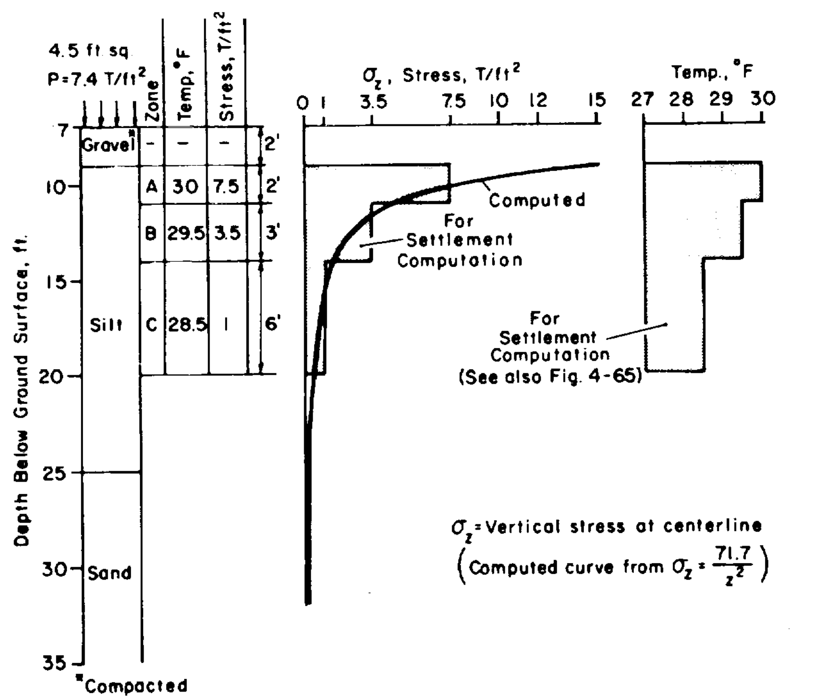

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Figure 4-66. Conditions for creep analysis.

Table 4-8. Settlement Computations

$$\text{STRAIN, } \epsilon(t) = \left[\frac{\sigma t^{\lambda}}{\omega(\theta + 1^{\circ})^{k}}\right]^{1/m} + \epsilon_o{*}$$

Zone	Zone Thick. (H) (ft)	Soil Type	θ (°F)	Vert. Stress. σ_z (psi)		Strain Period of Loading t=25yr ≈ 2x10⁵hr	Deformation (ft)	Deformation (in)
A	2	Silt	2	104	$\left[\frac{104t^{.074}}{570(2+1)^{.76}}\right]^{1/.49} = \left[0.079t^{.074}\right]^{1/.49} = 0.0057\,t^{.151}$	0.036	0.072	0.864
B	3	Silt	2.5	49	$\left[\frac{49t^{.074}}{570(2.5+1)^{.76}}\right]^{1/.49} = \left[0.033t^{.074}\right]^{1/.49} = 0.0010\,t^{.151}$	0.0063	0.019	0.228
C	6	Silt	3.5	14	$\left[\frac{14t^{.074}}{570(3.5+1)^{.76}}\right]^{1/.49} = \left[0.008t^{.074}\right]^{1/.49} = 0.000053t^{.151}$	0.00034	0.002	0.024
						TOTAL =	0.093 ft	1.12"

Values for Constants From Table 4-V:

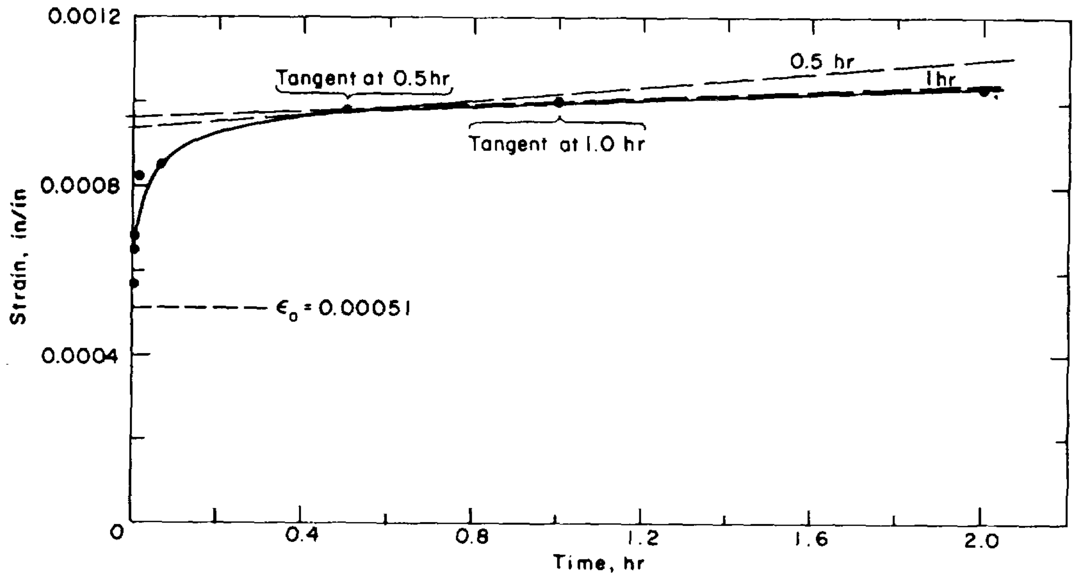
* The term, ϵ_o, can be neglected for estimating creep.

Silt

$m = 0.49$
$\lambda = 0.074$
$\omega = 570 \left[\text{psi (hr)}^{\lambda}\right] / {}^{\circ}\text{F}^{k}$
$k = 0.76$
$\theta = (32^{\circ} - \text{Temp. of soil})^{\circ}\text{F}$

If a settlement of 1 1/4" can not be tolerated, the footing should be enlarged to reduce the stresses or the foundation redesigned.

TOTAL SETTLEMENT IN 25 YR OF ABOUT 1 1/4 INCHES



$$\dot{\epsilon} = \frac{d\epsilon}{dt} = \text{slope of tangent at time, } t = 1 \text{ hr and } 0.5 \text{ hr}$$

$$t = 1 \text{ hr}, \dot{\epsilon}_1 = \frac{0.00105\text{-}0.000961}{2.0} = 0.000045$$

$$t = 0.5 \text{ hr}, \dot{\epsilon}_{.5} = \frac{0.00110\text{-}0.000933}{2.0} = 0.000083$$

$$M = \frac{\log(t_1/t_{0.5})}{\log(\dot{\epsilon}_{0.5}/\dot{\epsilon}_1)} = \frac{\log 1/0.5}{\log 0.000083/0.000045} = \frac{\log 2}{\log 1.8} = 1.11$$

$$\text{then } \epsilon = \dot{\epsilon}_1 \left(\frac{M}{M-1}\right) [t^{(M-1/M)} - 1] + \epsilon_0 = 4.5 \times 10^{-5} \times (10.1) [t^{0.099} - 1] + \epsilon_0$$

$$\epsilon = 4.55 \times 10^{-4} [t^{0.099} - 1] + 0.00051$$

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Figure 4-67. Unconfined compression creep test - data for Zone B. Specimen No. HAS-83. Applied stress 50 psi. Test temperature 29.5°F.

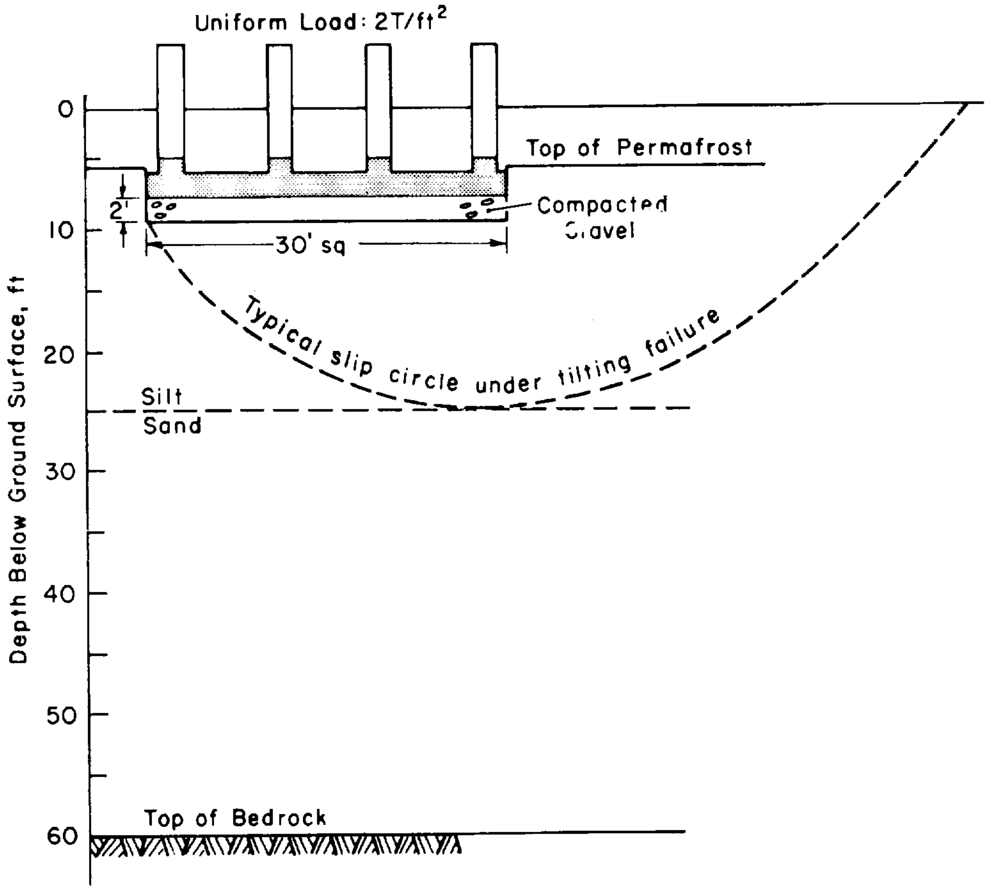

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Figure 4-68. Conditions for bearing capacity analysis of square raft (by CRREL). Desired/actor of safety = 2.0. A uniform load can be assumed for closely spaced individual footings or piers, or the area beneath piling.

qu = 1.3 c Nc + y D Nq + 0.4 yB N7

The value of "c" varies at different depths with the soil temperature and the soil type. A failure surface can be assumed as shown in figure 4-68 and a "weighted" value for "c" used, e.g.

$$c_{wt} = \frac{\sum_1^n c_n \times ARC_n}{\sum_1^n ARC_n}$$

c_n = cohesion for zone n

ARC_n = arc length contained in zone n

A conservative procedure is to use the smallest cohesion of the soil zones involved. Neglecting internal friction (N_λ = 0)

$N_c = 5.7$

$N_q = 1.0$

$D = 7$

$$q_u = 1.3\,c(5.7) + \frac{115(7)(1.0)}{2000}$$

$$q_u = 1.3\,c\,(5.7) + 2000$$

$$q_u = 7.41\,c + 0.40$$

Using cohesive stress of 2.0 T/fr' for silt at 30°F (from table 4-4 in paragraph 4-4, neglecting safety factor):

$$q_u = 7.41\,(2.0) + 0.40 = 14.82 + 0.40 = 15.22\ \mathrm{T/ft^2}$$

$$FS = \frac{15.22(30x30)}{2(30x30)}\ 7.61\ (>2)$$

This is very safe, but creep settlement may govern.

Making creep settlement estimate. Using charts from TM 5-818-1[5] for uniform load (based on Boussinesq equations) the stress distribution beneath the centerline of the footing (δ_z) is tabulated in table 4-9 and shown in figure 4-69 (δ_z = increase in the stress in the soil — neglect weight of soil).

Table 4-9. Stress Distribution Beneath the Uniformly Loaded Area.

Loaded Area.

a. Under center, b=15 ft.

z Depth below footing, ft	$\frac{b}{z}$	f(m,n) from chart	Stress, T/ft^2 a_z = 4xf(m,n) x 2T/ft^2
60	0.25	0.027	0.22
50	0.30	0.038	0.30
42.8	0.35	0.048	0.38
37.5	o.40	0.06	0.48
33.3	0.45	0.072	0.58
30.0	0.5	0.084	0.67
20.0	0.75	0.136	1.09
15.0	1.0	0.175	1.40
10.0	1.5	0.216	1.73
7.5	2.0	0.232	1.86
6.0	2.5	0.24	1.92
4.28	3.5	0.245	1.96
3.75	4.0	0. 247	2.00

b. Under corner, b=30 ft.

z Depth below footing, ft	$\frac{b}{z}$	f(m,n) from chart	Stress, T/ft^2 a_z = 4xf(m,n) x 2T/ft^2
60	0.5	0.085	0.17
50	o.6	0.107	0.21
42.8	0.7	0.128	0.25
37.5	0.8	0.146	0.29
33.3	0.9	0.162	0.32
30.0	1.0	0.175	0.35
20.0	1.5	0.216	0.43
15.0	2.0	0.232	0.46
10.0	3.0	0.244	0.49
7.5	4.0	0.247	o.49
6.0	5.0	0.248	0.50
4.28	7.0	0.250	0.50
3.75	8.0	0.250	0.50

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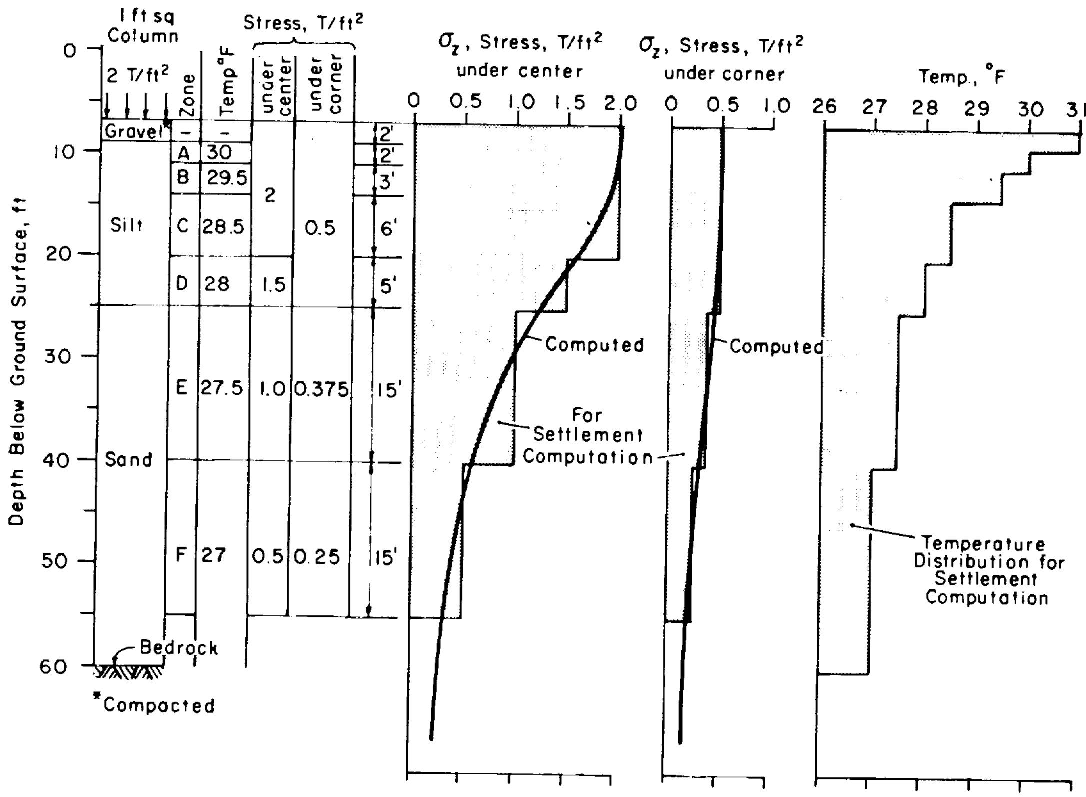

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Figure 4-69. Frozen soil column - diagrams of temperature and stress distribution.

Creep settlement deformations are computed using the stress and temperature distributions shown in figure 4-69. The height of the column of soil to be analyzed is determined by the stress and temperature distributions. The height of the column is assumed to extend to a depth where the magnitude of the stress is so small that the contribution to creep deformation below this point can be neglected. The cross-sectional area of the soil column is taken as the load area (i.e., the area of the footing). The column of soil used in the settlement estimate is shown in figure 4-69. The computed temperature distribution from table 4-7 and the computed stress distribution below the center of the foundation from table 4-9 are shown as smooth curves in figure 4-69. The rectangular shaded areas represent the approximate stress and temperature distributions used in the settlement computations. Table 4-10 shows the detailed computations for the settlements at the center and at the corner of the foundation at the end of 25 years, assuming it is flexible. If this differential settlement is not tolerable it will be necessary to increase the area of the footing or change the configuration or type of foundation. Of course, the revised design will require repetition of the settlement computations.

4-8. Piling.

a. General. Piling offers many advantages where construction is on frost-susceptible foundation soils in areas of deep seasonal frost penetration or permafrost with high ice content". Pile foundations can be constructed with minimum disturbance to the thermal regime and can isolate the structure from the seasonal heave and subsidence movements of the annual frost zone and from at least limited degradation of the permafrost. The structure load is transferred by the pile to depths where soil supporting strength remains relatively stable through the life of the structure. General information on engineering and design of pile structures and foundations is contained in EM 1110-2-29062[20] and TM 5-818-1/AFM 88-3, Chapter 7[5], and the references cited therein.

b. Pile types. Pile materials may be timber, concrete, or steel. Composite piles may be profitably used to pro

Table 4-10. Settlement Computations (by CRREL)

$$\text{STRAIN } \epsilon(t) = \left[\frac{\sigma t^{\lambda}}{\omega(\theta + 1^{\circ})^{k}}\right]^{1/m} + \epsilon_o{}^{*}$$

Zone	Zone Thick. (H) (ft)	Soil Type	Degree Freez. θ°F	Vert. Stress σ_z Ctr.	Vert. Stress σ_z Corner	STRAIN $\epsilon(t)$	Strain (Period of Loading $t=25yr=2\times10^5 hr$)	Deformation Center (ft)	Deformation Corner (ft)
A	2	Silt	2	28	7	$\left[\frac{28t^{.074}}{570(2+1)^{.76}}\right]^{1/.49} = \left[0.0213t^{.074}\right]^{1/.49} = 0.00038t^{.151}$	0.0024	0.0048	$\times(7/28)^{1/.49} = 0.00026$
B	3	Silt	2.5	28	7	$\left[\frac{28t^{.074}}{570(2.5+1)^{.76}}\right]^{1/.49} = \left[0.0190t^{.074}\right]^{1/.49} = 0.00030t^{.151}$	0.0019	0.0057	$\times(7/28)^{1/.49} = 0.00034$
C	6	Silt	3.5	28	7	$\left[\frac{28t^{.074}}{570(3.5+1)^{.76}}\right]^{1/.49} = \left[0.0156t^{.074}\right]^{1/.49} = 0.00021t^{.151}$	0.0013	0.0078	$\times(7/28)^{1/.49} = 0.00046$
D	5	Silt	4	21	7	$\left[\frac{21t^{.074}}{570(4+1)^{.76}}\right]^{1/.49} = \left[0.0108t^{.074}\right]^{1/.49} = 0.00010t^{.151}$	0.0006	0.0030	$\times(7/21)^{1/.49} = 0.00032$
E	15	Sand	4.5	14	5.2	$\left[\frac{14t^{.35}}{5500(4.5+1)^{.97}}\right]^{1/.78} = \left[0.0005t^{.35}\right]^{1/.78} = 0.00006t^{.448}$	0.0142	0.213	$\times(5.2/14)^{1/.78} = 0.060$
F	15	Sand	5	7	3.5	$\left[\frac{7t^{.35}}{5500(5+1)^{.97}}\right]^{1/.78} = \left[0.0002t^{.35}\right]^{1/.78} = 0.000018t^{.448}$	0.0043	0.065	$\times(3.5/7)^{1/.78} = 0.041$
							TOTAL =	0.299' 3.6"	0.102' 1.2"

* The term ϵ_o can be neglected for estimating creep.

Values for constants from Table 4-V.

Silt

$m = 0.49$
$\lambda = 0.074$
$\omega = 570\ [psi(hr)^{\lambda}]/{}^{\circ}F^{k}$
$k = 0.76$

Sand

$m = 0.78$
$\lambda = 0.35$
$\omega = 5500\ [psi(hr)^{\lambda}]/{}^{\circ}F^{k}$
$k = 0.97$

Estimated Total Settlement in 25 yrs.

At center of loaded area	=	3.6
At corner of loaded area	=	1.2
Differential Settlement	=	2.4"

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vide a large surface area and holding capacity within the permafrost but with a small circumference and area exposed to heaving forces within the annual frost zone. Composite piles employing wood as either the upper or lower member are not recommended unless there is no possibility that frost heave forces will act on the pile, because of the difficulty of providing a joint capable of resisting high tension forces under the effects of heaving. The type of pile selected will depend on initial costs, shipping costs, installation method, load levels, resistance to corrosion, difficulty in splicing or cost and difficulty of providing installation equipment, labor availability and other factors. Displacement piles, which densify or force aside a relatively large volume of soil as they are driven, can be used only in thawed soils.

(1) Timberpiles. Timber piles are normally less expensive than other types, easy to handle and normally readily available in lengths from 30 to 70 feet. Timber piles frozen into saturated permafrost soils are durable for centuries. Their structural characteristics are discussed in paragraph 2-6. The maximum allowable average compressive stresses on the cross section of round or square timber piles are given in TM 5-818-1/AFM 88-3, Chapter 7 [5].

(2) Steel piles. H-piles and pipe piles are the most useful types of steel piles, although box sections and angles have been used. Pipe or shell piles filled with concrete or sand may be used in some designs to provide high load capacity. Pipe piles that are capped or closedended at the bottom may be installed in premade holes but cannot be driven in permafrost as displacement piles. Open-ended steel pipe and H-piles can be driven in great lengths, can be readily cut off or made longer and can carry high loads. The average compressive stress on steel pipe and H-piles under the design load should not exceed 9000 psi[120]. For steel shells less than 1/10 in. in thickness, no contribution to bearing capacity from the shell should be credited. For steel shells 1/10 inch thick or greater, stress should not exceed 9000 psi[120]. Laboratory investigations should be made of the soils and water to which steel piles will be exposed to determine if corrosion will be a problem[183,184]. If corrosion protection is required, steel piles should be protected by a coal-tar preservative over a lead based primer from the pile cap to not more than 5 feet below the long term permafrost table. No corrosion protection is required, nor should it be used, below the latter depth in frozen soils. Reduction of the potential adfreeze bond may be expected as a result of shear failure in the coatings applied to the pile surface[23]. Any portion so coated extending below the permafrost table should be discounted in computing bearing capacity or frost heave resistance.

(3) *Concrete piles*.

(a) Concrete piles should not be used under conditions where frost heave forces may produce tensile stresses sufficient to crack the piles and expose the reinforcing steel to corrosion. Steel in lightly reinforced and lightly loaded concrete piles may be stretched and substantially by frost heave forces, causing multiple cracking The upward forces may be double the design loadings, causing complete stress reversal. Therefore, if the piles will be subject to heave forces, careful analysis should be made to insure that the amount of reinforcing steel in combination with structural loading is sufficient to prevent cracking. Pretensioned precast piles may be advantageous.

(b) Cast-in-place concrete piles should not in general be used in permafrost because of the hazards of either thawing the permafrost or freezing of the concrete. When such piles may be required in special cases, approval of HQDA (DAEN-ECE-G), WASH DC 20314 or HQUSAF/PREE, WASH, DC should be obtained based on field test evidence.

(c) The average compressive stress on any cross section of a pile should be in accordance with requirements in TM 5-818-1/AFM 88-3, Chapter 7 [5].

(d) Precast piles may be round, square, multisided or double T's. Handling, transporting and cutoff of concrete piles may be relatively expensive.

(4) *Special types of piles*.

(a) To assist freezeback at time of installation and to increase the degree of thermal stability under service conditions during period of below-freezing weather, several special types of metal pipe piles may be considered. Such piles, called as a group thermal piles, including self-refrigerated piles, serve not only to carry structural loads, but also to remove heat from the ground surrounding the embedded portion of the pile, and move it upward to the surface where it is dissipated to the atmosphere. In some cases only the heat removal function may be needed.

(b) In the two-phase system which operates on an evaporation-condensation cycle, analogous to a steam heating system with gravity condensate return, the pile is charged with propane, carbon dioxide or other suitable evaporative material"'. Evaporation of this material by heat flow from the ground and its condensation in the portion of the pile exposed to cold winter temperatures above the ground surface provides the heat transfer mechanism. Finned radiation surfaces are commonly employed above ground. Condensate returns by gravity to the liquid reservoir at the lower end of the pile.

(c) In the single-phase system or liquid convection cell, analogous to a gravity hot water heating system, the pile is completely filled with a suitable nonfreezing fluid, and heat is moved upward from the ground by a natural circulation induced by a density gradient of fluid resulting from the temperature dif ference between the exposed top and the embedded bottom of the pile in winter[145].

(d) Both the above systems are self-initiating systems. They automatically cease operation when air temperatures become warmer than those around the lower part of the pile. They are intended to require no operating attention once properly installed. Patents have been obtained for proprietary systems in both areas.

(e) In a fluid forced circulation system, analogous to a circulated hot water heating system, a pump is used to circulate a nonfreezing liquid or gas through the pile and through a surface heat exchanger exposed to the atmosphere in winter, or through a mechanically refrigerated heat exchanger without regard to season. The refrigerated fluid is usually introduced at the bottom of the pile through a central inner pipe and then flows upward in contact with the inner wall of the pile. The same objective may be gained by circulation of refrigerated fluid through tubing attached to the exterior of the pile for artificial freezeback as described in d (3) below. The refrigeration may be temporary for the purpose of achieving initial freezeback during construction or may be permanent where required by the design thermal conditions. Direct circulation of cold winter air through a pipe pile by a simple fan has been shown to be very effective in principle", but it is possible the circulation may be rapidly blocked by frost accumulation if the system is under unfavorable conditions; this alternative is therefore not recommended for permanent construction in the present state-of-the-art.

(f) Thermal piles which depend on seasonal dissipation of heat to the cold winter air may be expected to show a temperature drop effect to about 8 to 10 feet radius by the time above-freezing air temperatures arrive in the spring. At least a small depression of ground temperature should remain through the fall from this effect, in order for the piles to achieve their long range purpose. Piles which use artificial refrigeration can keep the ground refrigerated in all seasons.

(g) An essential requirement of all closed system thermal piles is freedom from leakage. Leakage of liquid containing antifreeze may seriously and permanently degrade the ad freeze bond strength of the pile. Leakage of gas through fittings, welds or porous metal may quickly make the system inoperative. All such installations must therefore be very carefully pressure tested to detect any leakage. All thermal pile units using liquefied petroleum products must be constructed to meet the standards of the National Fire Protection Association NFPA 58-1979 Standard for the Storage and Handling of Liquefied Petroleum Gases and the ASME Section VIII-1977 Pressure Vessels Division 1. Methods of computing heat transfer by thermal piles are presented in e below.

c. Pile emplacement methods. Many of the earlier foundations in permafrost used local timber, installed in steam or water-thawed holes to depths which rarely exceeded 20 feet. Normally the piles were pushed or lightly driven into the pre-thawed holes. They often required restraint to prevent flotation. Pile length and spacing tended to be dictated by structural requirements of the building rather than by pile bearing capacity or by the ability of the permafrost to accept the heat introduced during the installation. Pile installation techniques now utilize modern drilling or driving techniques and effectively minimize permafrost thermal disturbance. Installation methods are determined by ground temperatures, the type of soil, the required depth of embedment, the type of pile, and the difficulty and cost of mobilizing the required equipment and personnel at the site. Installation methods which may be considered are as follows: (1) Installation in dry augered holes.

(a) Holes for the piles may be drilled in permafrost by using earth augers with bits specially designed for frozen ground[37,74,80,133,211]. Two-feet-diameter holes can be advanced at rates up to about I ft/min in frozen silt or clay depending on type of bit, ground temperature and size of equipment. Holes up to 4 feet or more in diameter can be drilled readily in such soil. Advantages include minimum required effort and equipment, accurate positioning and alignment, and accurate control of hole dimensions and therefore of heat input into the permafrost. Drilling is easily carried out under winter conditions when the frozen ground surface permits ready mobility, without the problems of handling water or steam under freezing air temperatures. However, the method is not likely to be feasible in coarse, bouldery frozen soils. The holes may be drilled undersize and wood or pipe piles may be driven into the holes. However, more commonly the holes are drilled oversize and soil-water slurry is placed in the annular space around the pile and allowed to freeze back as described below.

(b) The mixture of soil and water (slurry) used to fill the annulus between augered hole and pile can often transfer the imposed pile loads to the surrounding frozen soil more effectively than the original in-situ soils. The auger-slurry method is easily adapted to production methods the auger followed by a pile-placing crane, followed by a slurry crew. If surface water is not entering the augered holes which would require relatively quick action, they may be covered with plywood, or by other methods, until pile placement is convenient. This will permit each crew to work independently. Silt from borrow or from the pile hole excavation can be used for the slurry, as well as gravelly sand, sand or silty

sand. Clays are difficult to mix and place and do not achieve good adfreeze strength values. Gravel, unsaturated soils or only water (ice) should not be used for backfill. Concrete should not be used to backfill around piles in drilled or augered holes in permanently frozen ground. Organic matter (peat) should not be permitted in the material used for preparation of the slurry. Strength and creep properties of slurries are discussed in more detail in f(l) below.

(c) Slurries are normally prepared in portable concrete mixers by adding sufficient water to bring the slurry within a prescribed range of water content. The mix water temperature may be warm or even heated when using frozen cuttings. When thawed borrow material is used water is chilled by the addition of snow and/or ice. When mixed, slurry temperature should under no circumstances exceed 40 °F. The slurry should have the consistency of 6 inches slump concrete, the consistency being specified and monitored by field inspectors using a calibrated container from which the acceptable range of wet unit weights can be quickly determined. Normally the mixing crew can reproduce the desired consistency quite easily after establishing the appearance and viscosity during the first or second batch. Methods for control of components may be similar to those used in concrete batching. Based on past experience, continuous inspection of the mixing operation should be made to insure that no more water than necessary to saturate the soil and to produce the desired consistency of the slurry is used and that temperature of the mix is kept low.

(d) The slurry is normally placed by direct backfilling by wheelbarrow on small jobs or by the use of concrete buckets with cranes on large jobs. Tremie pipes or direct pumping may be advantageously used on some jobs but during cold weather, operations are often a source of major trouble because of freezing. Normally the pile after being centered and plumbed in the hole is backfilled with the slurry in one continuous operation. It is very important that the material be vibrated with a small diameter concrete spud vibrator and rodded to ensure that there are no bridging and no voids left along the pile, which can often happen when backfilling around cylindrical piles, especially if tremie pipe is not used. Care should be taken during placement of the slurry to avoid moving or bending the pile by placing the slurry too fast from only one side. Small form vibrators may also be attached directly to the pile to aid in uniform flow around the pile during backfilling and to hasten consolidation of the backfill. Rapping the sides of the pile with a sledge hammer also aids the backfilling and consolidation.

(e) Timber piles quite often float or rise up when backfilled with silt-water slurries and must be restrained or weighted. Anchoring timber piles may also be accomplished by backfilling only a portion of the hole depth and permitting the lower portion to freeze back before completing the backfill. Care must be taken not to coat the pile or hole wall with the slurry above the level of the backfill since it will freeze in place, and tend to prevent complete filling of the voids when the final lift is placed later. To avoid flotation no additional slurry backfill should be placed around the pile until the pile has frozen solidly in place.

(f) Backfilling may also be accomplished by filling the hole with silt-water slurry, just prior to placing the pile, sufficiently to bring the slurry to the surface when the pile is inserted to proper depth. Piles thus placed are more difficult to plumb and position but when such factors are not critical this method is much faster. H-piles are easily placed by this method; timber and closed-end pipe piles have a strong tendency to float. Such prior backfilling of holes should not be attempted when using sand backfills unless the piles are to be driven into the unfrozen slurry.

(g) Normally no attempt is made to completely clean or to bell the bottom of augered holes, some loose cuttings always remaining. Bottom portions of holes drilled too deep can be backfilled as necessary with sand or gravel slurry while the pile is suspended in the hole. The pile may be dropped a short distance to compact the cuttings or backfill. Piles (except H-piles) should not be placed closer than 1 inch to the walls of augered or drilled holes.

(h) To minimize the amount of heat to be introduced into the ground by the slurry, the annular space is made only just large enough to allow the slurry to be efficiently placed and vibrated with a small diameter concrete vibrator. Vibration is needed to insure that the space will be completely filled without air entrapment. Except for H-piles, a minimum of about 3 inches oversize in diameter is required to permit use of a 1-inch-diameter vibrator in the annular space. Additional allowance is usually made because of the difficulty of achieving exact centering of the pile in the hole and because of pile irregularities such as lack of straightness. A hole size 4 to 6 inches larger than the pile diameter commonly has been used, but it should be kept as small as practical for the particular situation.

(2) *Installation in bored holes.* Holes for the piles may also be made by rotary or churn drilling, or even by drive coring under some conditions, using various bits, drive barrels, etc., and removing frozen materials with air, water and/or mechanical systems[55,74,153]. Procedures are otherwise the same as for dry augered holes. By proper selection of equipment, any type of frozen ground may be handled. Use of water or warm air for removal of cuttings may introduce undesirable quantities of heat into the permafrost and must be carefully controlled.

(3) *Installation by driving.*

(a) Conventional or modified pile driving procedures including diesel and vibratory hammers may be used to drive open-ended steel pipe and H-section to depths up to 50 or more feet in frozen ground composed of silty sand or finer-grained soils, at ground temperatures above about 25°F[48,49,55,74,169]. Some experience indicates that, under favorable conditions, heavy pipe and H-sections can be driven into ground at lower temperatures, as described in paragraph 2-6. Although the pile is heated by the driving action and a thin zone of soil may be thawed at the soil/pile interface, the amount of heat thus introduced into the permafrost is usually negligibly small and freezeback is normally complete within 15 to 30 minutes after completion of driving. Because no drilling of pile holes is required, because no slurrying is involved, and because total installation equipment can be minimal, this installation technique can be even simpler than the dry augered hole technique. However, the procedure is limited to steel pipe and H-piles, and it is necessary to make certain that sufficient driving energy is available to reach the depths of penetration needed for bearing and to resist frost heave. Templates should be used to assure accurate placement of piles.

(h) The smallest H section to be considered for driving in frozen soil should normally not be smaller than 10BP42 and the rated hammer energy should not be less than 25,000 ft-lb. When piles are driven with conventional or vibratory hammers, advance should be continuous, since there is a negligible amount of heat evolved, and stops longer than 5-10 minutes can permit freezeback to the extent that resumption of driving will be impossible, or possible only after a prolonged period of heavy driving. Necessity for stops to weld on additional lengths should be avoided. The use of chemicals, jetting or steaming should not be permitted during driving although the pile may be preheated (particularly lower half) as it enters the ground to minimize side friction during pile penetration.

(4) *Installation by steam or water thawing*. Until the early 1950's, piles were traditionally installed in permafrost by prethawing the ground at the pile locations by steam points before driving. An alternative was water thawing. However, these techniques have the disadvantage of introducing so much heat into the ground that freezeback may be almost indefinitely delayed. This involves not only the volume of permafrost thawed by the steam or water, which is difficult to control, but also the warming of a large volume of surrounding frozen material. The result may be failure to develop adequate bearing capacity and/or progressive working of the piles out of the ground by frost heave with consequent damage to supported structures. Many such failures have occurred. Therefore, steam or water thawing should not be used in any area where the mean annual permafrost temperature is greater than 20°F and may be used in colder heat input into the ground if alternative methods of installation are not feasible.

d. Freezeback of conventional piles.

(1) General.

(a) Piles and anchors in permanently frozen ground attain their holding capacities only after they are frozen solidly in place. Pile-supporting capacity in permafrost is dependent primarily on the strength of the adfreeze bond between the permafrost and the pile surface. The strength of the bond is a function of temperature and is at its lowest and most critical value in the fall and early winter when permafrost temperatures at the levels in which the piles are supported are at their warmest. Therefore, any unnecessary transferal of heat from the structure to the foundation will tend to have an adverse effect on the supporting capacity. In far northern areas the reserves of supporting capacity and stability may be so large that small variations in heat input to the foundation will be of little consequence; in marginal permafrost areas, however, the effect of even small unanticipated heat inputs may be extremely critical.

(b) Freezeback of slurry or otherwise thawed soil surrounding piles must be assured before imposing any load upon the pile. Thus, in addition to the time required to install the pile, the construction schedule depends on the time required for freezeback of the pile. If foundation piles are installed well in advance of the structural construction or if the permafrost temperature is well below freezing, there may be adequate time available for natural freezeback by permafrost. If the construction time is short and the work is to be continuous or if the permafrost temperature is warm, use of artificial refrigeration or thermal piles may be required. Thus, the foundation thermal conditions may determine both the design and the method of construction to be employed.

(c) In order to measure the rate and effectiveness of freezeback of slurried piles, and to permit monitoring of subsequent foundation performance, thermocouple or thermistor assemblies should be attached to representative piles or thermistor assemblies should be attached to representative piles in the foundation. When artificial freezeback is employed using tubing attached to the pile exterior, the temperature sensors should be placed midway between tubes where freezeback will be longest delayed. A control thermocouple assembly installed at an adjacent undisturbed area and in equilibrium with the ground temperatures is essential for comparison with temperatures at the piles. To avoid conflicts all monitoring equipment should be provided, installed and observed by the Government.

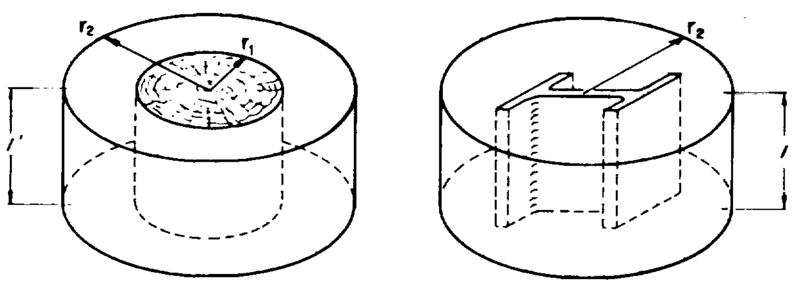
(d) Since fine-grained soils tend to freeze or thaw at temperature levels depressed below 32 °F,

theoretical computations of freezeback and pile spacing involving such materials should use the actual freezing or thawing temperatures, rather than 32 °F. This is especially important when normal permafrost temperatures exceed about 29 °F to 300°F. If either the slurry material or the permafrost is other than silt, sand or gravel in which practically all the moisture freezes at the nucleation temperature, the volumetric latent heat of the slurry and/or the volumetric heat capacity of the permafrost within the range of the placement, freezeback, and thermal adjustment temperatures will have to be determined by test. The freezing characteristics of the soil can be ascertained in the laboratory by generating a cooling curve with time[160] or by calorimetry; they can also be inferred from study of natural in-ground temperatures during seasonal freeze-thaw flux. In important projects, test piles instrumented with thermocouples or other temperature indicating devices should be used to verify the freezeback potential of the permafrost prior to actual construction. Since test conditions can seldom be identical with the actual construction ground temperatures, the results must usually be analytically transformed to the construction conditions.

(2) *Natural freezeback*.

(a) Soil-water slurries placed in drilled or augered holes introduce heat, which in natural freezeback is conducted into the surrounding permafrost. The heat content of the water, soil, and pile can be computed, if the water content and dry unit weight of the slurry are known or determined by experiment. Slurry placed at temperatures from 32 °F to 40 °F, under normal conditions, has a total heat content above 32°F of less than 200 Btu/ft^3; this is small enough to be merely approximated and added to the latent heat in the computation procedures outlined below.

(b) The latent heat per foot of pile length is computed by the equations shown in figure 4-70. Note that the latent heat is governed only by the volume of slurry, the water content (w) and dry unit weight (d). Thus, the heat input into the permafrost can be minimized by control of the dimensions of the annulus (which is also a function of the type of pile) and by selection and control of the slurry. Simple comparisons of the amount of latent heat per unit volume of slurry can be made using the following equation, assuming all the water freezes:



$$Q = \pi L (r_2^2 - r_1^2) w \gamma_d \quad \text{or} \quad Q = L(\pi r_2^2 - A) w \gamma_d$$

where L = Latent heat of water

r_2 = Radius of hole

r_1 = Radius of pile

A = Cross sectional area of H-pile

w = Water content, percent dry weight/100

γ_d = Dry unit weight of slurry

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Figure 4-70. Latent heat of slurry backfill[134].

$Q = L\,w\,\gamma d$ (Equation 8)

where

L = latent heat, 144 Btu/lb of water

w = water content, expressed as decimal

λd = dry unit weight, lb/ft^3.

Examples are as follows:

For w = 80 %, λd = 53, Q = 144 (0.80)(53) = 6100 Btu/ft^3

For w = 40%, λd = 80, Q = 144 (0.40)(80) = 460 Btu/ft^3

For w = 19%7, λd = 109, Q = 144 (0.19)(109) = 3140 Btu/ft^3

(c) Thus, a silt slurry or one with an excess of water may introduce considerably more heat than a sand slurry or one in which the amount of water is carefully controlled.

(d) When the slurry moisture content is carefully controlled, the slurry will retain relatively uniform characteristics after freezing. However, if the slurry has an excess of water, consolidation of the soil component may result in separation of excess water from the slurry. Even while freezeback is proceeding inward from the wall of the hole, bridging of the soil in the relatively narrow annular space may result in formation of essentially soil-free slugs of water between masses of consolidated slurry. Because of their high heat content, the water inclusions will freeze back more slowly than the consolidated slurry. If the water inclusions occur within the annual thaw zone, they may thaw and escape to the surface in subsequent seasonal thawing, even though frozen initially, and settlement of the overlying slurry may then occur, requiring backfilling of the resulting depression around the pile.

(e) Freezeback of slurry proceeds primarily from the wall of the hole inward toward the pile. If the pile itself is below freezing temperature, freezing may also occur from the pile surface outward, particularly if the pile is a pipe type open at the surface to admit air at low temperatures. When the slurry is composed of frost susceptible fine-grained soil, multiple small ice lenses will form during freezeback; these are oriented vertically, parallel to the wall of the hole. An annular layer of ice normally forms at the contact between the slurry and the wall of the hole, sometimes as much as an inch thick; this has no significant effect upon the pile bearing capacity. A similar layer of ice may also form on the surface of the pile; because the ability of the material at this contact surface to endure tangential shear stresses is controlling in determining allowable pile bearing capacity, the occurrence of such an ice layer may be significant. If piles of any type are placed during below freezing air temperatures by the slurry method it should be assumed, unless evidence can be presented to the contrary, that an ice layer, however thin, will form on the surface of the pile. In such cases it will be necessary to assume allowable tangential adfreeze bond stress corresponding to whichever is the weaker, under the critical permafrost design temperature, of pure ice or consolidated slurry. (For further discussion of strength of ice vs. strength of frozen soil, see f(1) below.) This problem will not arise when piles are installed in permafrost by driving.

(f) While it may be possible to avoid formation of an ice layer on the pile during installation at low temperatures by use of non-frost-susceptible slurry a number of problems may cause difficulty in achieving this result. In the first place, a slurry conforming to the common definition of non-frost-susceptible material presented in TM 5-818-2[6] is not necessarily completely non-frost-susceptible. That criterion assumes that a certain low level of frost susceptibility is tolerable in pavement applications and is based upon freezing rates experienced under pavements. Validity of the criterion for slurry freezeback conditions has not been investigated. Also, under field conditions it may be difficult to insure that some contamination with fines may not occur from contact of the above-freezing slurry with the wall of the drilled hole, or from other sources.

(g) Some test pits made around piles which have been in place a year or more have shown a layer of ice on the pile extending from the ground surface through the annual thaw zone, attributed to segregated freezing of seasonally thawed moisture. Such an ice layer limited to the annual thaw zone is not significant in terms of the pile bearing capacity.

(h) Measurements on 8-inch steel pipe piles exposed to the atmosphere and sunlight above the ground in a region of borderline permafrost have shown that thawing may typically reach several inches below the top of permafrost immediately adjacent to the pile surface at a location where the permafrost table is 3.8 feet below the ground surface. An additional few inches may have especially low tangential adfreeze bond strengths. Therefore, the assumed effective length of embedment in permafrost of all properly installed dark-surfaced piles exposed to sunlight should be reduced by a nominal 15 inches. No reduction is required for piles completely shaded, shielded, or painted a highly reflective white, regardless of type of pile. For piles improperly installed, as by uncontrolled steam thawing, no valid guidance can be given.

(i) Knowledge of ground temperature with depth is essential to estimate the freezeback time and overall effect of the installation on the permafrost. Plots showing seasonal variation of depths of isotherms in the ground or plots of temperature with depth may be used to select the optimum installation period for rapid freezeback. Available methods of computing natural freezeback of piles in permafrost assume the slurried pile to be a finite cylindrical heat source inside a semi-

infinite medium, with a suddenly applied constant temperature (32°F) source which dissipates heat only in a radial direction into frozen ground of a known initial temperature[14,134]. The general solution for the natural freezeback problem, based upon latent heat content of the slurry, is shown in figure 4-71.

(j) To determine the time required for freezeback at different permafrost temperatures, it is easier to use a specific solution similar to that shown in figure 4-72, prepared from the general solution. The specific solution is computed using the known or estimated thermal conductivity and volumetric heat capacity of the permafrost and the diameter of the hole to be used. As previously noted, allowance for any heat content of the slurry above 32°F may be made with sufficient accuracy by adding this heat content to the volumetric latent heat, Q; this assumes that the placement temperature of the slurry is controlled below about 40°F. The specific solution in figure 4-72 clearly demonstrates the effect of latent heat of slurry and initial permafrost temperature on the time required for freezeback.

While not specifically shown in figure 4-72, months may be required to freeze back slurries of 10,000 Btu/ft3 or more volumetric heat capacity at permafrost temperatures between 30" and 32°F. Under otherwise identical conditions, a sand-water slurry in 28 °F permafrost could freeze back in 2 or 3 days while a siltwater slurry would take 10 to 11 days; however, in 31.5 °F permafrost, a freezeback time of about 16 days would be required for the same sand-water slurry and about 130 days for the same silt-water slurry. Thus, for given pile spacings careful selection of the pile type, hole size, slurry material, and installation season, plus careful control of water content, can substantially reduce the amount of heat which must be absorbed by the permafrost and the time required for freezeback.

(k) The preceding general and specific solutions

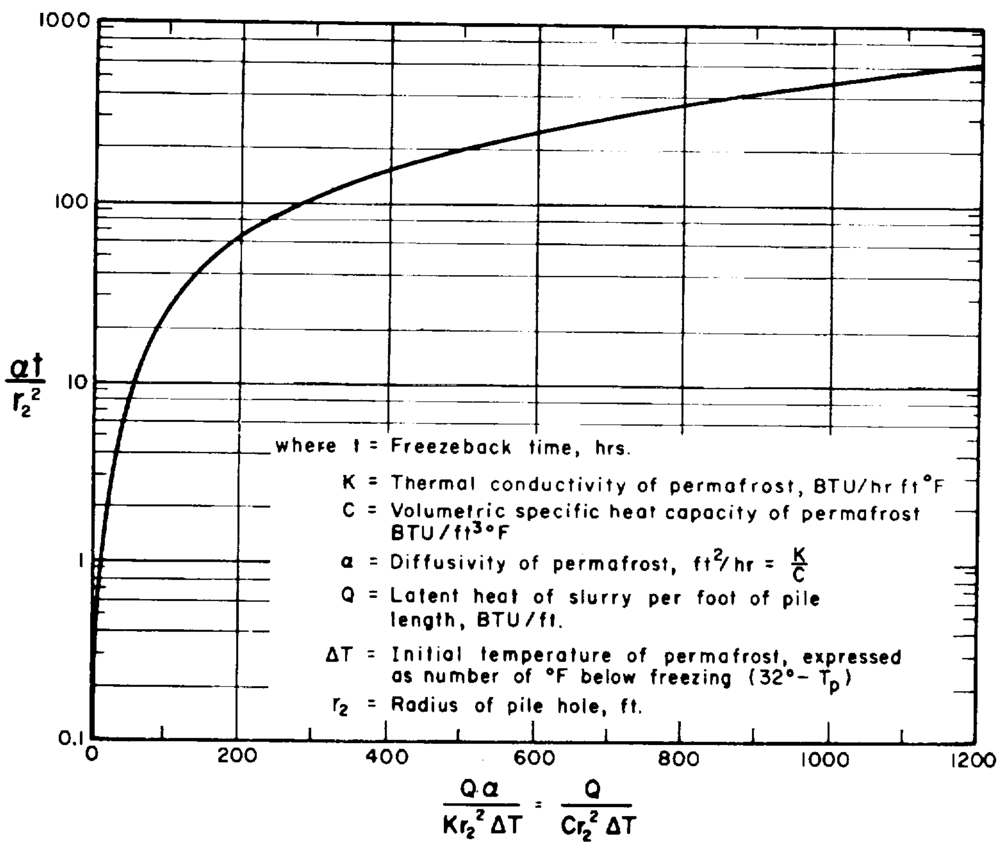

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Figure 4-71. General solution of slurry freezeback rate[134].

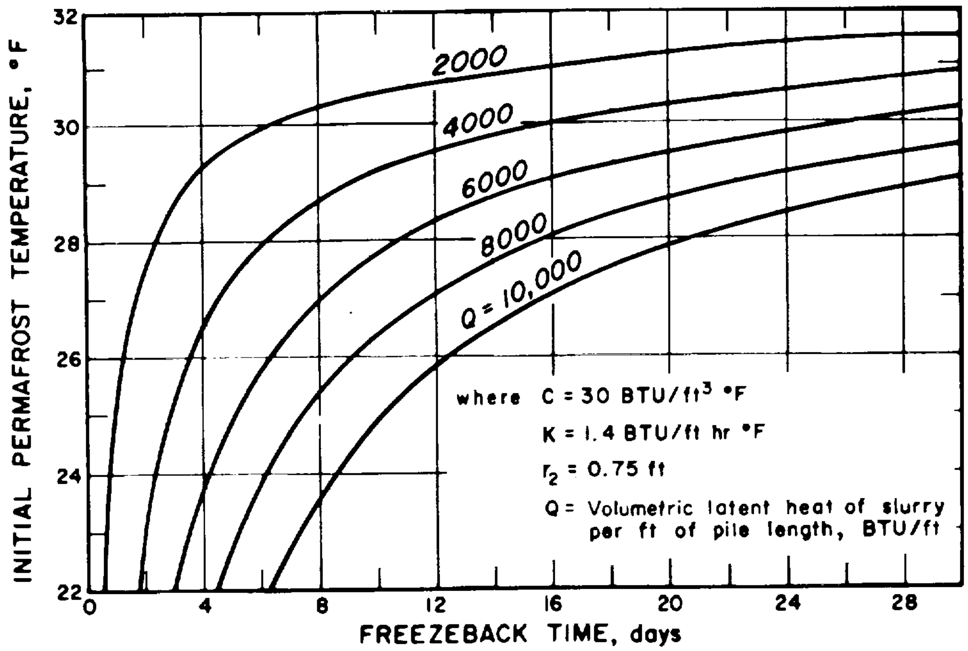

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Figure 4-72. Specific solution of slurry freezeback rate[134].

assume the slurry heat to be conducted only in a horizontal radial direction. The actual heat paths during summer and winter are approximately as shown in figure 4-73. However, while freezeback time may be increased or decreased by deviations of heat flow from the horizontal, and adjustment of the assumed "effective" temperature of the permafrost may sometimes be necessary to allow for this effect, the increase in freezeback time which may be caused by proximity between adjacent piles is a more dominant consideration ((4) below), everything else being equal.

(3) *Artificial freezeback*.

(a) When ground temperatures are too warm or the amount of heat introduced is too great to accomplish natural freezeback of slurry within the planned construction period, artificial refrigeration must be used to accomplish the desired freezing of the backfill. The artificial freezing may be accomplished by circulation of refrigerating fluid through longitudinal or spiral steel or copper tubing attached to the pile, or by use of thermal piles as described in e below. Brine or glycol solutions and ambient air have all been used as the circulating fluid. However, the use of propane or other refrigerants of similar characteristics has been found to be the most efficient and economical. Propane has the disadvantage of flammability. The refrigerant may be circulated through piles either individually or in series, using a portable compressor, as shown in figures 4-74 and 4-75. The size of the compressor depends on the number of piles to be frozen and the amount of heat to be removed from the slurry. Pipe size, exposure area per foot of pile length, and rate of circulation are other

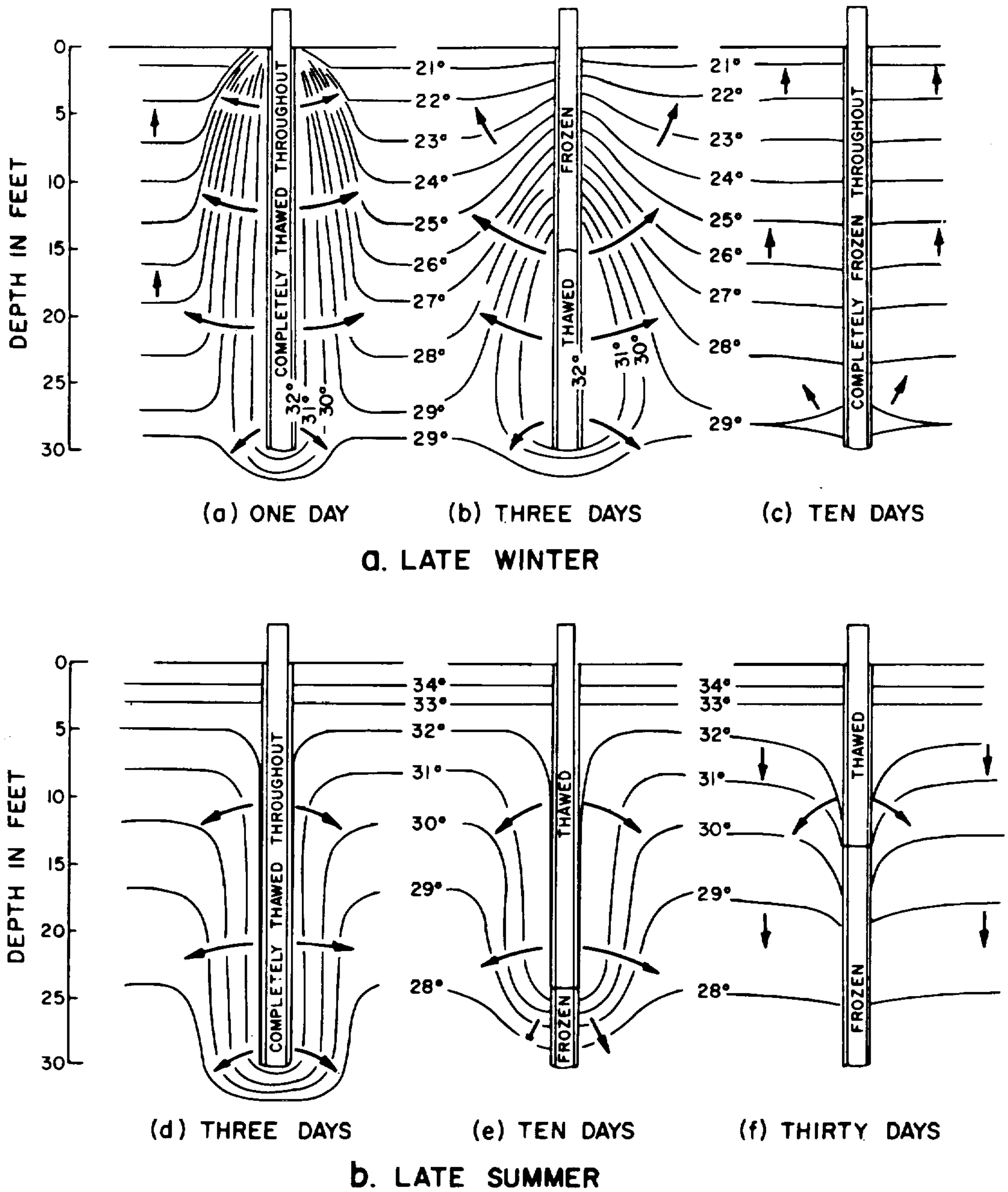

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Figure 4-73. Natural freezeback of piles in permafrost during winter and summer[134].

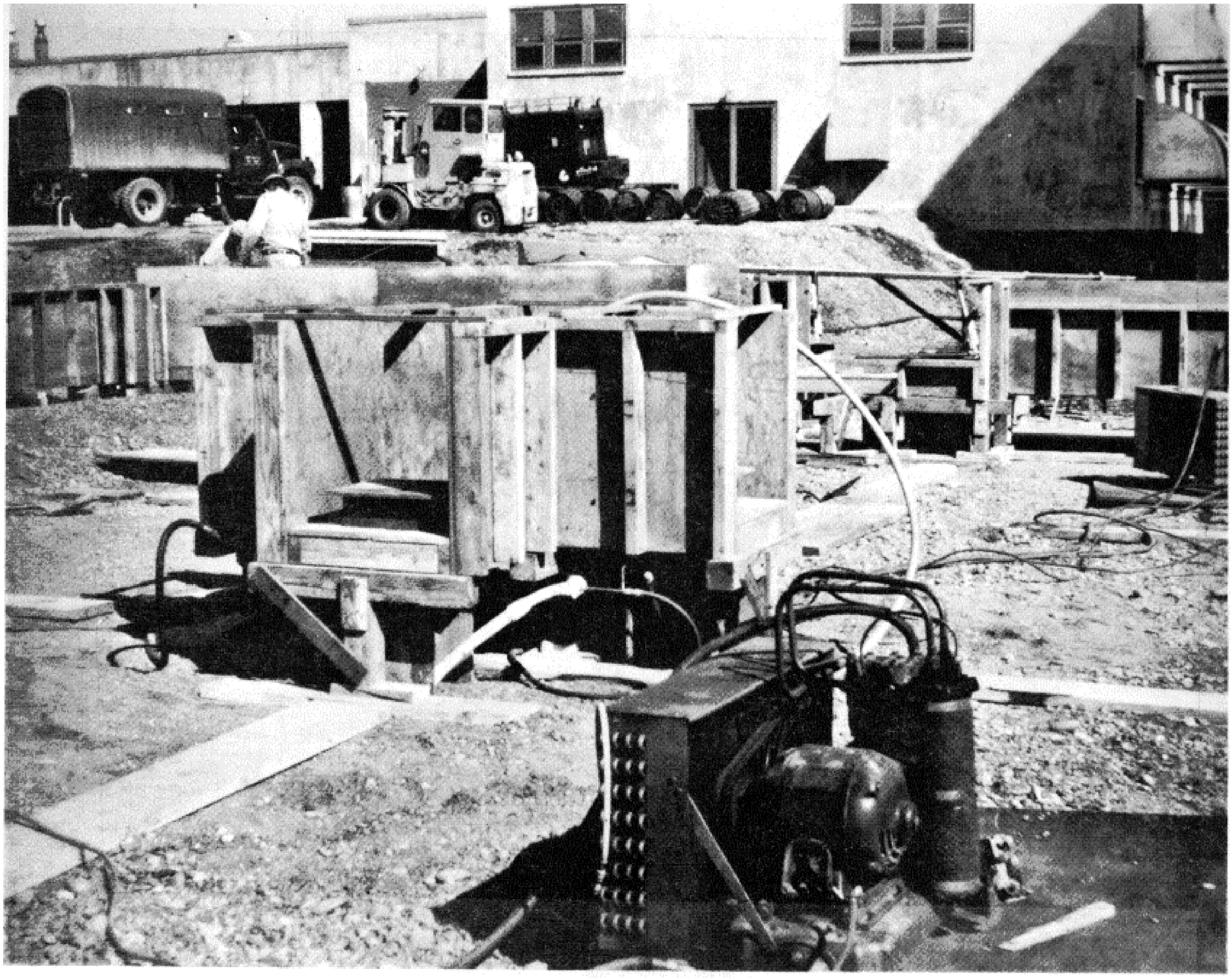


Figure 4-74. Compressor for artificial freezeback of piles.

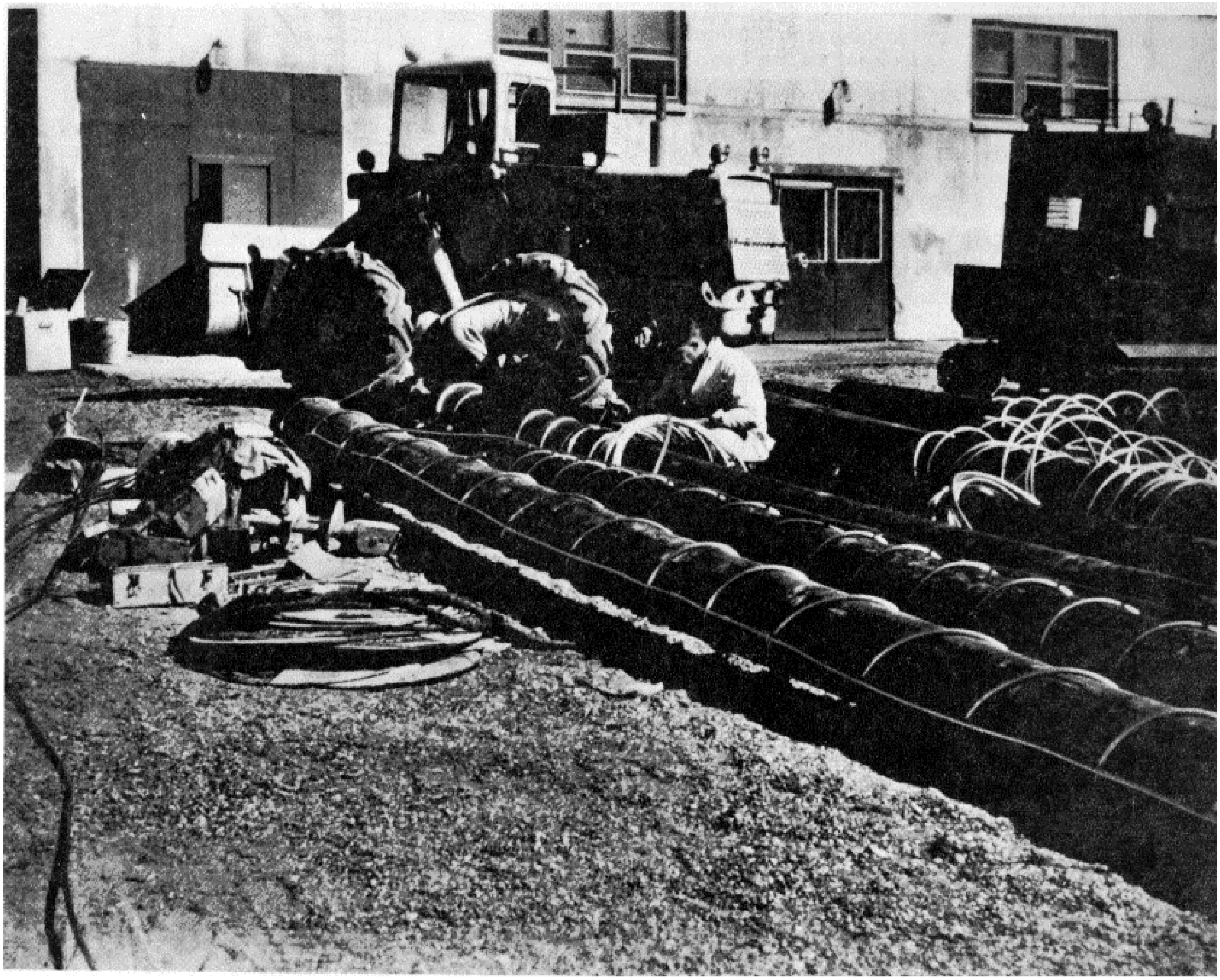


Figure 4-75. Refrigeration coils on timber piles for artificial freezeback.

parameters influencing the rate of freezeback. Method of computing freezeback time for a given refrigeration capacity is given in TM 5-852-6/AFM 88-19, Chap 6[14]. By limiting the time between slurry placement and start of refrigeration to less than a day, the heat gain by the surrounding permafrost can be minimized. Establishment of a proper freezeback criterion is very important. Very low temperatures can be produced at a given moment close to the refrigerant tubing but the soil may still be unfrozen several inches away. Therefore, the duration of the refrigeration period should be established as that which when suspended for 24 hours will produce frozen ground temperature at the critical freezeback location no greater than the normal ground temperature at that position. The controlling depth where the freezeback is slowest is often about 20 feet, but may be anywhere between the top of permafrost and the bottom of the pile. Temperatures may also need to be monitored simultaneously at two or more different depths for control. When the required period of refrigeration has been established for one or more monitored piles, it is thereafter necessary to monitor freezeback on only a limited number of selected production piles for spot check purposes. Careful records should be kept of the freezing plant and ground temperature observations.

(b) Unless refrigeration is to remain permanently in operation, refrigerant tubes on the piles should be filled with arctic engine oil chilled to below existing ground temperatures and sealed when the refrigeration period is completed. Should refrigeration be required at a later date, oil can be removed and the refrigeration system reactivated with minimum effort; in the interim, the oil provides protection against corrosion and ice blockage.

(c) Internal refrigeration of pipe or other hollow piles can be accomplished by use of automatic or forced circulation thermal piles as described in e below.

(d) Artificial freezeback can also be accomplished by the evaporation and expansion from the liquid or solid state of gases vented to the atmosphere. Such gases include nitrogen, carbon dioxide, propane and other similar materials. Dry ice has, for example, been placed in pipe piles to effect rapid freezeback. Such means of rapid freezeback are usually too expensive for use in large installations but can be effectively employed in small installations.

(4) Relation of pile spacing to freezeback.

(a) The spacing of piles is normally based on structural requirements of the floor system or on the need to provide a sufficient number of piles to support relatively large concentrated vertical loads. Consideration of pile spacing in early phases of the structural design may make it possible to provide sufficient distance between piles so that in permafrost areas natural freezeback can be utilized to effect substantial savings.

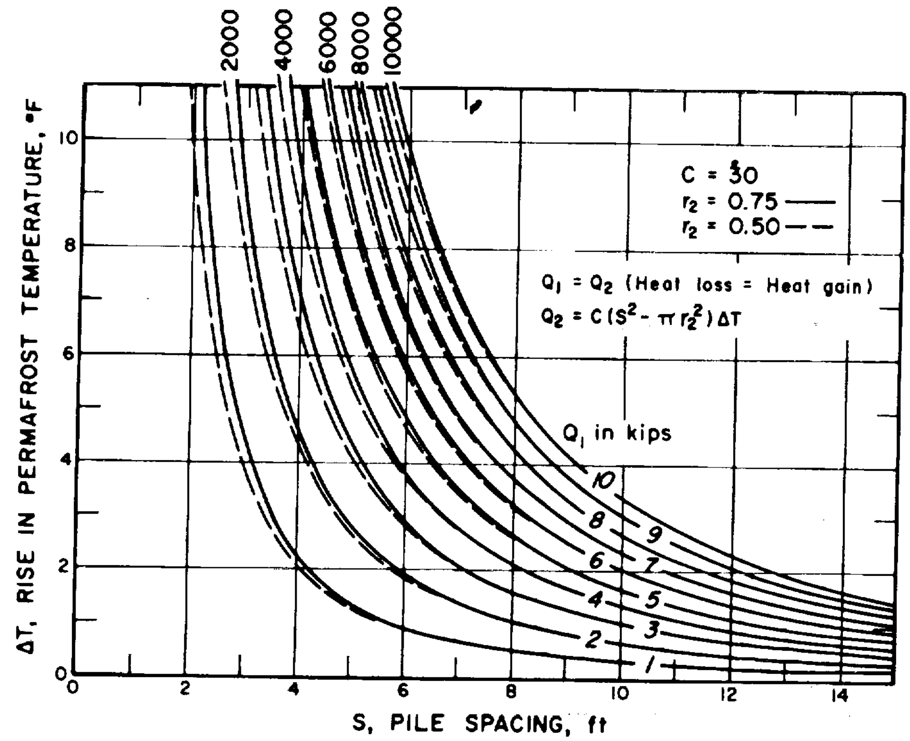
(b) Driven piles, which introduce negligible amounts of heat, have no critical spacing other than that required to facilitate movement and operation of the driving equipment or that introduced by possible group action effect in the foundation.

(c) Slurried piles, however, produce an overall rise in permafrost temperature[134]. The effect of pile spacing on permafrost temperature rise at different slurry heat values is illustrated in figure 4-76. The relationship between temperature rise and slurry heat, as influenced by the volumetric heat capacity of the permafrost and spacing, is given by the equations in the figure. If the rise in permafrost temperature (AT) indicated by figure 4-76 should exceed the difference between the freezing point and the initial permafrost temperature (T_f-T_p) the permafrost can not freeze more than the amount of slurry water which will raise the permafrost temperature to its thawing point. Heat exchange cannot occur when permafrost and slurry are at the same temperature. The remaining slurry will not freeze until the surrounding permafrost becomes colder. Actual freezing or thawing temperatures of the materials should be used in the analysis where these differ from 32°F.

(d) No factor of safety is incorporated in the pile spacing effect equations and chart presented in figure -I 4-76. Therefore, it is essential that temperature indicating devices be required as part of the design to verify freezeback during construction (again taking into account the freezing characteristics of the soil).

(5) *Period of installation.*

(a) The natural freezing rate of slurried piles is primarily dependent on initial ground temperatures of the permafrost and the spacing of the piles[134,137]. As illustrated in figures 1-1 and 1-3, the coldest ground temperatures are experienced in the spring. In areas of marginal permafrost, permafrost temperatures are so high that there is insufficient "reserve of cold" in the permafrost to insure natural freezeback of slurried piles except in spring (approximately February, March, April and May). If slurried piles must be installed in marginal permafrost areas at other times of the year, artificial refrigeration must be employed to insure slurry freezeback. If freezeback is not completed before the refreezing of the annual frost zone starts in the fall, and frost heaving occurs, the adfreeze bonding required for support of the design load may never be achieved. On the other hand, steel piles may usually be installed in fine-grained permafrost soils by driving at any time of the year in these areas without freezeback problems. Where permafrost temperatures are below about 25°F, installation by driving will usually be impractical, thus - requiring a slurried type of installation. However, at



where Q = Latent heat of slurry, BTU/ft of pile length
C = Volumetric heat capacity of permafrost, BTU/ft³ °F
S = Pile spacing, ft.
r_2 = Radius of pile hole, ft.
ΔT = Rise in temperature of permafrost, °F.

Figure 4-76. Influence of slurry on temperature of permafrost between piles[134].

such temperatures problems of slurry freezeback are greatly reduced. If augering is accomplished prior to start of thaw in spring or early summer, the holes normally require no casing and are not subject to filling with melt water. Augering cold frozen soils requires no additional power; in fact cold frozen cuttings are easier to displace at the surface when spin-removed and are easier to shovel or scoop up for removal or use in the slurry. Snow on the ground surface may be partially or completely removed, but compacted snow offers a good working surface which helps to protect vegetative cover. Compaction of snow greatly reduces the insulation value of the snow cover, permitting colder temperatures to develop in late winter.

(b) Most construction contracts involving slurried piles are awarded so as to permit the contractor to install piles in late winter or spring, thereby allowing the work to progress throughout the summer, with the structure being closed in against weather by late fall. If the pile installation is done in summer and fall the work tends to be hampered by ground water, sloughing of soils in holes, slow freezeback, and a loss of equipment mobility on the ground surface unless a granular mat is placed. Even though the ground surface may be frozen in fall, a residual thaw zone may still be present well into the winter, possibly requiring a casing to seal off ground water and sloughing soil. Early winter is also unfavorable because air temperatures are often uncomfortable, there is minimum daylight for work, and ground temperatures in the permafrost are near their warmest.

(c) On the other hand, piles which are to be driven into frozen soil can be best driven in the early winter. In the spring, open-ended pipe piles are somewhat more difficult to drive than in the early winter because of colder permafrost temperatures, the presence of the seasonal frost layer and the tendency of the soil plug developed inside the pipe to wedge against the sides. This wedging action can develop to an extent

that the effort required to advance the pile is similar to that required for a closed-end pipe pile and may make it impossible to advance the pile at all. Normally H piles require less additional effort in the spring as compared to fall.

(d) The design engineer should therefore carefully consider the period of installation with respect to mobilization, transportation, work and equipment efficiency, ground water and soil problems when augering as well as the attendant freezeback time. Since the problems associated with the installations at different periods of the year will be reflected in the quotations.

e. Heat transfer by thermal piles.

(1) Two-phase thermal piles.

(a) As previously noted, the two-phase system operates on an evaporation-condensation cycle wherein the vapor condenses on the inner walls of the pipe pile and flows down the pipe walls to mix with the liquid phase. The requirement for spontaneous operation of the device is that the temperature in the upper reaches of the interior wall must be colder than the saturation temperature of the vapor. The selection of the refrigerant should consider such factors as its vapor pressure, vapor density, and flammability. A refrigerant having a low vapor pressure at a given temperature will tend to minimize the leakage potential and to simplify sealing. A high liquid density at a given temperature will tend to increase the gravity forces which remove the liquid condensate after its formation on the upper walls of the pile. Although the thermodynamics of the internal pipe refrigerant are important, particularly the thermal resistance of the condensate film which varies in thickness along the interior pipe wall, the governing resistance (exclusive of the freezing soil surrounding the pile) may be assumed to be the air boundary layer on the pile's exterior surface. This is particularly true for conditions of heat transfer from the exposed portion of the pile by essentially natural convection. On this assumption, rate of heat transfer to the exterior from the exposed surfaces of a two-phase thermal pile may be estimated using the following equation: further, assuming that; the pipe and soil are in intimate contact along the entire buried portion; the pipe relies solely upon heat dissipation from its vertically oriented surface i.e., no horizontal piping connections at the surface) and; the pile is of sufficient diameter so that the upward vapor flow and downward condensate flow do not impede their mutual development:

ΔT = TV - Ta, °F

T_V = refrigerant vapor temperature, °F

T_a = ambient air temperature, °F

A_l = surface area of pile exposed to air per lineal foot, ft^2

L_a = length of pile exposed to air, ft

(b) The addition of fins to the pile improves its heat transfer capability. An indication of this improvement can be determined for a *unit length of pile per fin by:*

$$q_o = 2\,\delta\,K_1\,N\,\Delta T\,\tanh\,(Nw + \sqrt{Nu}) \qquad \text{(Equation 10)}$$

where

$\sqrt{Nu} \leq 1/2$

δ = half thickness of fin, ft

K_1 = thermal conductivity of fin material, Btu/ft hr °F

$N = \sqrt{h_c/K_1\,\delta}$

w = width of fin, ft

Nu = Nusselt number $= \dfrac{h_c\,\delta}{K_1}$

(c) For the case of unfinned piles, natural convection (no wind), and assuming that turbulent conditions generally prevail, the equation $q = hc\,A\,\Delta T = hc\,A\,(T_v - T_a)$ (equation 9) is modified by introducing:

$$h_c = C_1\,K_{air}\quad a\,(T_v - T_a)^{1/3} \qquad \text{(Equation 11)}$$

where:

C_1 = constant = 0.13 (vertical cylinder)

K_{air} = thermal conductivity of air at temperature $T_m = 1/2\,(T_v + T_a)$, Btu/ft hr °F

$a = g\,\beta\,\rho\quad c_p/\mu\,K_{air}$, $1/ft^{3\,o}F$ (see table below)

where:

a is determined for the mean temperature condition, Tm

g = acceleration of gravity, ft/sec^2

0 = coefficient of expansion for air, $I/°F_{abs}$

p = air density, lb/ft'

c_p = specific heat of air at constant pressure, Btu/lbm °F

J = absolute viscosity of air, Ib_m/ft hr

Thus, for unfinned piles, natural convection, no wind:

$q = 0.13$ Lair a1/3 A $(T_v - T_a)$ 4/3

(d) For the case of unfinned piles, forced convection, induced either naturally by wind or mechanically, the surface transfer coefficient is modified and the equation is:

$$q = h_c\ A\ (T_v - T_a) = \frac{K_{air}}{D}\ (0.82)\ \frac{VD\rho}{\mu}\ 0.585\ A(T_v - T_a)$$

where:

V = wind velocity, ft/hr

D = outer diameter of pile, ft

iValues of factors p ,p , a and K_{air} for various values of T_m are given below:

T_m (°F)	ρ (lb/ft^3)	μ (Ib_m/ft hr)	a ($1/ft^3$ °F)	K_{air} (Btu/ft hr)
32	.0807	.0417	2.21×10^6	.0140
20	.0827	.0408	2.50×10^6	.0136
0	.0863	.0394	3.00×10^6	.0131
-10	.0882	.0387	3.47×10^6	.0129
-20	.0902	.0380	3.93×10^6	.0126
-30	.0923	.0373	4.53×10^6	.0123
-40	.0945	.0366	5.21×10^6	.0121
-50	.0968	.0358	5.74×10^6	.0118
-60	.0992	.0351	6.47×10^6	.0115
-70	.1018	.0344	7.24×10^6	.0113

(f) Computation of heat transfer during freezeback. The thermal pile may be used to accelerate freezeback of the slurry in a preaugered hole. It would thus tend to supplement the in-situ permafrost's freezeback capability. As noted in figure 4-72, a wet slurry of 10,000 Btu/ft in a relatively warm permafrost at 28°F would require about 20 days to freeze back naturally for the numerical values assumed in that example. An indication of the reduction in freezeback time afforded by the thermal pile is developed below.

(g) Example. Assuming a pile length of 20 feet below ground surface (i.e., including the annual frost zone), this represents a total of 200,000 Btu's of latent heat to be-removed during slurry freezeback. Further, assuming that an unfinned pile is placed during the late fall when the average daily air temperature is 20°F and no wind exists, the following estimation of freezeback under the thermal pile mechanism may be made:

$q = 0.13\ K_{air}\ a^{1/3}$ Al $(T_v - T_a)$ 4/3

Tv = 32°F (assumed to be the temperature of slurry during freeze-up)

Ta = 20°F

Kair = 0.0138 Btu/ft hr °F for Tm = 1/2 (Tv + Ta)

= 26 F (from table)

a = 2.35 x $106/ft^3$ °F for Tm = 26°F (from table)

From a pile of 1 foot nominal diameter, the cooling area, At, is (rx 1.062) = 3.33 ft'/lineal foot. Thus:

$q = (0.13)\ (0.0138)\ (2.35 \times 10^6)\ 1/3\ (3.33)\ (32\text{-}20)\ ^{4/3}$

= 21.8 BtuAin ft hr

(h) The freezeback time, relying exclusively on the thermal pile effect, for a pile length, La, of 4 feet exposed to the air is:

$$\frac{200{,}000\ 9}{21.8 \times 4 \times 24} = 96 \text{ days}$$

(i) This assumes that none of the slurry heat is extracted by the surrounding permafrost, which, of course, is not the case. During the 20-day period required to naturally freeze back the slurry, heat removal via the thermal pile effect is (21.8 x 4x 20 x 24) = 42,000 Btu. This represents 2100 Btu/lineal ft of pile. Again referring to figure 4-72 and using a volumetric latent heat of slurry of (10,000 2100) = 7900 Btu per lineal foot, it is noted that 7900 Btu can be removed in about 15.5 days. Thus, the thermal pile will influence freezeback over a shorter time than 20 days and by successive approximations the appropriate freezeback time is established. In 16.5 days, the thermal pile extracts 1740 Btu/ft and about (10,000 1740 =) 8260 Btu/ft is dissipated into the permafrost in the same time interval (fig. 4-72). This represents a reduction in freezeback time of about 17 percent.

(j) Had the air temperature averaged 0 F, rather than 20°F, the thermal pile heat removal rate would have increased to 81 Btu/lineal ft hr and the overall freezeback time would have been reduced to about 10 days (a 50% reduction in time). It should be noted that these calculations assume that heat is also extracted by the pile from that portion of the slurry in the annual frost zone. If the pile is placed at the end of the winter, the annual frost zone will be at a lower temperature than the permafrost and thus more slurry heat will be removed per linear foot by the surrounding ground in the annual frost layer than is the than is the case in the permafrost zone. Thus, the procedure will tend to estimate the freezeback benefit of the thermal pile somewhat conservatively. However, the opposite situation develops should the pile be placed at the end of the summer period when the active zone is above freezing. At this time a large percentage of the pile's heat sink ability is used to extract heat from the annual frost zone.

(k) The above calculations neglected any consideration of the thermal benefit of finning the pile. A 1/4-inch-thick, 6-inch-wide fin of 4-feet length would dissipate 6.2 Btu/lin ft hr at 20 F air temperature and 20.7 Btu/lin ft hr at 0°F. The use of four fins would have the effect of essentially doubling the heat transfer rate from a cylindrical surface. The use of six fins, a number commonly used, would reduce the freezeback time from 16.5 and 10 days to 12 and 6 days for ambient air temperatures of 20 °F and 0 °F respectively.

(l) The above example assumed that the air would be quiescent during the freezeback period; the effect of an average wind of only 2 mph is considered next:

$$q = K_{air} \frac{(0.282)}{D} \left(\frac{VD\rho}{\mu}\right)^{0.585} A_\ell (T_v - T_a)$$

where:

ρ = 0.0817 lb/ft^3 at 26°F (from table in (e) above)

μ = 0.0412 lb/ft hr at 26°F (from table in (e) above)

V = 2 mph x 5280 = 10569 ft/hr

substituting:

$$q = \left(\frac{0.0138}{1.062}\right) (0.282) \left(\frac{10{,}560 \times 1.062 \times 0.0817}{0.0412}\right)^{0.585} (3.33)\ (32-20)$$

= 51 Btu/lin ft hr for T_a = 20°F.

(m) The freezeback time is reduced from 20 to about 12 days, representing a reduction of 40% from the natural freezeback time without thermal pile assistance. Had the temperature averaged 0 °F, the time would be reduced by about 60%o to 7-3/4 days. These results are tabulated below:

	Freezeback Time (Days)			
	Without Fins		With Fins (6)	
Wind	20°F	0°F	20°F	0°F
0 mph	16 1/2	10	12	6
2 mph	12	7 3/4	--	--
5 mph	9 1/2	5	--	--

Natural Freezeback Time 20 days (Permafrost at 28 °F)

(n) As indicated by these calculations, the amount of surface area presented to the cold outside air is critical and thus it is essential that snow (which is a rather good insulator) not be allowed to impede heat transfer from the pile. Heat transfer by the emission of long-wave radiation from the pile will accelerate the heat transfer process while absorption of solar radiation tends to retard heat transfer. The use of high albedo paint to reflect the incoming solar radiation is a common practice.

(o) Computation of heat transfer in service. An indication of the magnitude of temperature depression below the mean temperature of the ground surrounding the pile is useful in appraising the potential adfreeze strength provided by the thermal pile. This problem is best solved by means of a finite difference approach utilizing a digital computer. However, some useful relationships can be obtained via a steady state analysis assuming that the mean ground temperature, T^g, remains unchanged at a distance of ten radii from the pile.

The heat removed by the pile is:

$$q = h_c A\ell (T_v - T_a) L_a$$

while the heat input from the soil is:

$$q = \frac{2.73\ K_s\ (T_g - T_v)\ L_s}{\log\ (r_2/r_1)} \qquad \text{(Equation 12)}$$

where:

h_c, A_ℓ, L_a, T_v, T_a were defined above

L_s = length of pile exposed to soil, ft

K_s = frozen soil thermal conductivity, Btu/ft hr °F

r_1 = radius of pile

r_2 = 10 r_1

T_g = ground temperature at r_2

These two heat flow rates are equal.

$$h_c\ A_\ell\ (T_v - T_a)\ L_a = 2.73\ K_s\ (T_g - T_v)\ L_s$$

Thus:

$$T_v = \frac{2.73\ K_s\ (L_s/L_a)\ T_g + h_c\ A_\ell\ T_a}{2.73\ K_s\ (L_s/L_a) + h_c\ A_\ell} \qquad \text{(Equation 13)}$$

For the sample problem above in which the average wind speed was 2 mph, it is estimated that the permafrost temperature would be depressed from 28 °F to about 19° at the pile/soil interface.

(2) Single-phase piles.

(a) As previously described, the single phase system, or convection cell, operates by virtue of a density gradient induced by temperature difference between the above-ground (exposed to air) and the below-ground portions of the pile. Such systems may use a confined liquid, or gas, or ambient air as the heat transfer medium within the pile. As the fluid extracts heat from the soil surrounding the pile, its density decreases, thereby causing the fluid to rise and be replaced by overlying cooler fluid. Heat exchange to the atmosphere is accomplished either through the pile wall for liquid systems or by

direct mixing for air systems. Successful operation of this concept requires use of plumbing within the pile which physically separates the warm and cold fluid columns. As air temperatures increase above ground temperatures, the convective process is stopped, thereby preventing induction of warm ground temperatures. It should be noted that when ambient air is used as the heat transfer medium, summer winds may cause undesirable air flow within the pile and necessitate the use of positive shutoffs.

(b) Although a potentially simpler, essentially unpressurized system, the thermal efficiency of the natural convection, single-phase pile is less than that for two-phase systems owing to the increased internal resistance associated with for the liquid-filled pile: primarily the mass flow of the liquid with some contribution due to the liquid side boundary layer thermal resistance in the portion of the pile exposed to air and for the ambient air-filled pile: the low volumetric heat capacity of the air at low rates. Some laboratory studies reported by Johnson"' indicated that for liquid-filled (both water-ethyl alcohol and trichloroethylene), 2-inch and 4-inch model piles, a 5° to 15 °F temperature difference between air and the material to be cooled was necessary to achieve any heat transfer over a range of air flow from 0 to 40 mph; this indicates the influence of inertia forces which must be overcome to permit development of fluid flow.

(c) There are few heat transfer field data available for this type of pile system at time of preparation of this publication.

(3) Forced circulation piles.

(a) In some cases it may be necessary to install artificial refrigeration pipe or tubing on the pile to accelerate slurry freezeback time and to have such refrigeration available in the event that permafrost temperatures rise to unacceptable levels after construction. The thermal pile technique is restricted to that period of the year when air temperatures are low and normally cannot be used to accelerate freezeback during the summer period. The following example shows calculations required to determine the amount of heat to be extracted from the ground.

(b) Example. It is assumed that the average volume of slurry backfill for a group of piles is 31 feet^3 each. The slurry is placed at an average temperature 40 °F and must be frozen to 23 °F. A silt-water slurry of 80 lb/ft^3 dry weight and 40 percent water content is used as backfill material, and an available refrigeration unit is capable of removing 225,000 Btu/hr. Calculate the length of time required to freeze back a cluster of 20 piles.

Volumetric latent heat of backfill

$L = (144 \times 80 \times 0.40) = 4{,}600$ Btu/ft^3

Volumetric heat capacity of frozen backfill

$C_f = 80\ [0.17 + (0.5 \times 0.4)] = 29.6$ Btu/ft^3 °F

Volumetric heat capacity of thawed backfill

$C_u = 80\ [0.17 + (1.0 \times 0.4)] = 45.6$ Btu/ft^3 °F

Heat required to depress the slurry temperature to the freezing point:

$45.6 \times 31\ (40 - 32) = 11{,}310$ Btu/pile

Heat required to freeze slurry:

$31 \times 4{,}600 = 142{,}600$ Btu/pile

Heat required to depress the slurry temperature from the freezing point to 23 °F:

$29.6 \times 31\ (32 - 23) = 8{,}260$ Btu/pile

Total heat to be extracted from the slurry:

$20(11{,}310 + 142{,}600 + 8{,}260) = 3{,}243{,}000$ Btu

Time required for artificial freezeback excluding allowances for system losses

3,243,000/225,000 = 14.4 hr (with losses allow 20 hr)

(c) The maximum operating temperature for the coolant is usually set 10 F below the desired in-situ permafrost temperature and the temperature rise in the system should be fixed at 5 °F or less. Thus, for this example, the maximum temperature is (23 - 10) = 13°F and a difference of 5 °F would place the lowest temperature at 8 °F. The coolant freezing point should be at least 10F below this minimum. If it is likely that the air temperature will fall below this freezing point during the refrigeration operation, then the low air temperature would establish the freezing point for the coolant.

(d) Allowing a temperature rise of 4°F in the refrigerant and selecting a 21 percent sodium chloride brine, the required circulation rate is:

$$\frac{225{,}000}{60 \times 0.799 \times 4 \times 1.169 \times 62.h/7.5} = 120.6 \text{ gpm}$$

(e) Using 3/4-inch black pipe on the pile, the rate of circulation is in the order of 1 ft/sec, which should be considered as an upper limit. This size pipe or tubing is most commonly used.

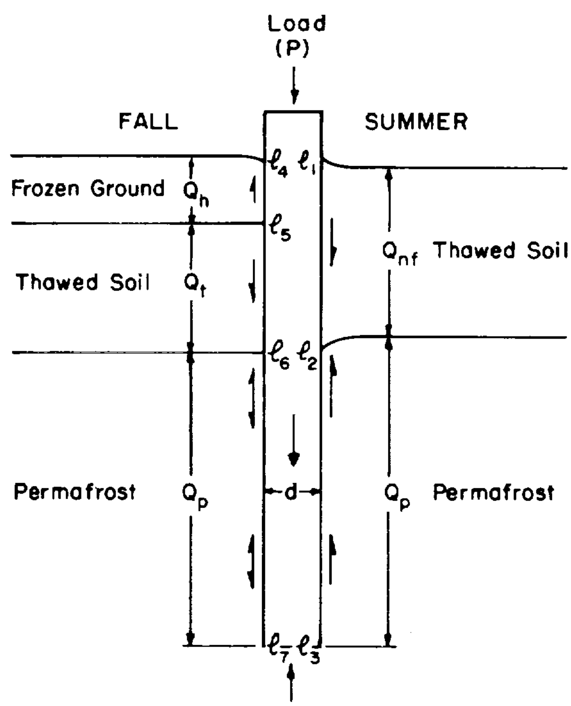
(f) The brine temperature will average 11°F which will result in a temperature difference between the surrounding soil of (40 - 11) = 29°F at completion of the refrigeration cycle.

(g) It is essential that temperature sensors be used to insure that proper freezeback rates and temperatures are obtained.

(h) Should the option of forced circulation of a gas or liquid within a closed metal pile be considered, the computation procedures outlined above may be adapted, the case being technically comparable to the freezing points used for stabilization of ground in construction or for stabilizing foundations experiencing permafrost degradation. However, so far as is known this approach has not been used except experimentally in conventional foundation bearing piles in North

America.

f. Design depth of pile embedment. Pile foundations must be designed for sufficient depth of embedment to support the imposed loads in adfreeze bond without objectionable displacement under the warmest ground temperatures expected, unless suitable end bearing on ice-free bedrock or other reliable strata can be obtained. The piles must also be capable of resisting the additional down drag of negative skin friction from consolidating fill or thawed foundation soils and must provide sufficient anchorage and tensile strength to prevent upward displacement and pile structural failure from frost heave forces in the annual frost zone in winter. The forces acting on a pile in permafrost during freezing and thawing seasons are shown in figure 4-77. During spring and early summer, piles have greatest potential bearing capacity in ad freeze bond because of the low permafrost temperatures during the period[133]. When the zone of annual thaw and freeze is in the process of refreezing, the extremely low temperatures in the frozen soil near the ground surface cause much higher adfreeze bond strengths in this area. During the same period of seasonal freezing, ground temperatures along the pile length in permafrost are at their warmest and the corresponding permafrost adfreeze strengths are at their weakest. Unless the pile is adequately embedded in the permafrost and capable of mobilizing sufficient resistance in adfreeze bond, the pile will heave if an up-



Point Bearing, Q_E

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Figure 4-77. Stresses acting on piling for typical permafrost condition[134].

ward frost heave thrust occurs exceeding the combined weight of the pile, load on the pile, negative skin friction in thawed zone(s) and the adhesion in the permafrost.

(1) *Friction piles.*

(a) Analytical considerations and methods of making preliminary estimates of bearing capacity of piles which develop their support in skin friction along their surfaces are discussed in this paragraph. Bearing capacity of such piles should be calculated for the ground conditions which exist in the most critical period of the year. In permafrost areas this will usually be late summer through early winter when permafrost temperatures at the depths of primary load support are at their warmest. Allowable downward load on piles supported in adfreeze bond in permafrost should be computed in accordance with equation 14 (see right-hand side of fig. 4-77):

$$Q_a = \frac{1}{FS}(Q_p \pm Q_{nf}) \qquad \text{(Equation 14)}$$

where

Q_a = Allowable design load on pile

FS = Factor of safety

Q_p = Maximum load which may be developed in tangential bond between pile and permafrost,

$$\int_{\ell_2}^{\ell_3} f_a dA_p$$

where

Ap = surface area of pile in permafrost

f_a = maximum tangential adfreeze bond stress which may be developed between frozen soil and pile, a function primarily of temperature, everything else being equal.

Q_{nf} = Maximum skin friction force from thawed soil on pile. Under normal summer conditions this will be a negative force, acting downward:

$$= \int_{\ell_1}^{\ell_2} f_s dA_t$$

where

A_t = surface area of pile in thawed soil

f_s = sum of unit friction and adhesion between thawed soil and pile

(b) As with footings, allowable loadings of piles in frozen ground are determined by creep deformation which occurs under steady loadings at stress levels well below the rupture levels measured in ordinary relatively rapid tests to failure. A creep deformation rate of only 0.01 in./day will result in 3.65 inches of settlement per year or about 3 feet in 10 years, which is wholly unacceptable for permanent type structures. Creep occurs in the adfreeze bond zone at the contact surface between the pile and the frozen ground and is attended by punching at the pile tip if the pile is overloaded. The stress-strain behavior of frozen soil in unconfined compression tests may be used to illustrate the deformation phenomena associated with support of loads on piles in frozen ground. Response of frozen silt to various conditions of loading is shown diagrammatically in figure 4-78 (the same relationships are represented in another form in fig 4-47). Elastic behavior is limited to a negligibly small portion of the stress-strain curve. Nonelastic deformation begins only a short distance from the origin and increases with increase in stress. At rapid rates of loading or at low temperatures, the stress-strain curves are relatively steep, relatively high stress levels are reached, and the deformation ends in brittle-type rupture, as shown by the two left-hand curves in figure 4-78. At slow rates of loading and warmer permafrost temperatures the curves are flatter, lower stress levels are reached, and deformation continues plastically to large strain values. If rate of applied strain is reduced at a point such as A, the stress intensity will tend to relax as indicated by curve AB to a stress level compatible with the new, lower rate of strain. In saturated, fine-grained frozen soils the peak and ultimate strengths tend to be virtually identical when loading rate is at a level producing extended deformation, as shown by the two right-hand curves in figure 4-78. Peak stresses higher than ultimate strength values are observed in saturated, granular frozen soils, even for slow rates of loading, but such soils less frequently require pile-type foundations.

(c) Figure 4-79 illustrates diagrammatically the manner in which load applied at the top of a pile is transmitted with time into relatively warm permafrost. For simplicity, load is assumed carried solely in skin friction in permafrost with zero load on the tip of the pile. Conditions are shown in figure 4-79 for three separate times after instantaneous application of load: t_1 represents a time immediately after load application, t_2 represents a time intermediate between t_1 and t_3 and t_3 represents time when complete stress-strain adjustment has occurred under the applied load. Immediately after load application, load transfer to permafrost is concentrated in upper sections of the pile. Load transfer to lower parts of the pile is at that time restricted because shortening of the pile in compression is restrain-

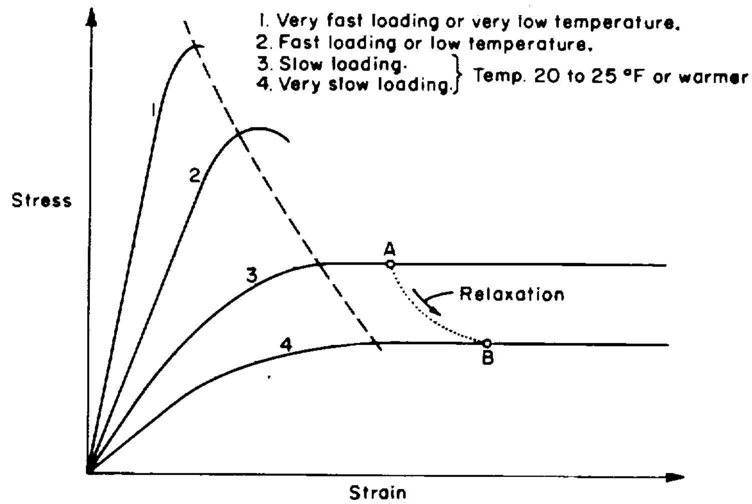

Figure 4-78. Response of frozen silt to loading conditions in unconfined compression (by CRREL).

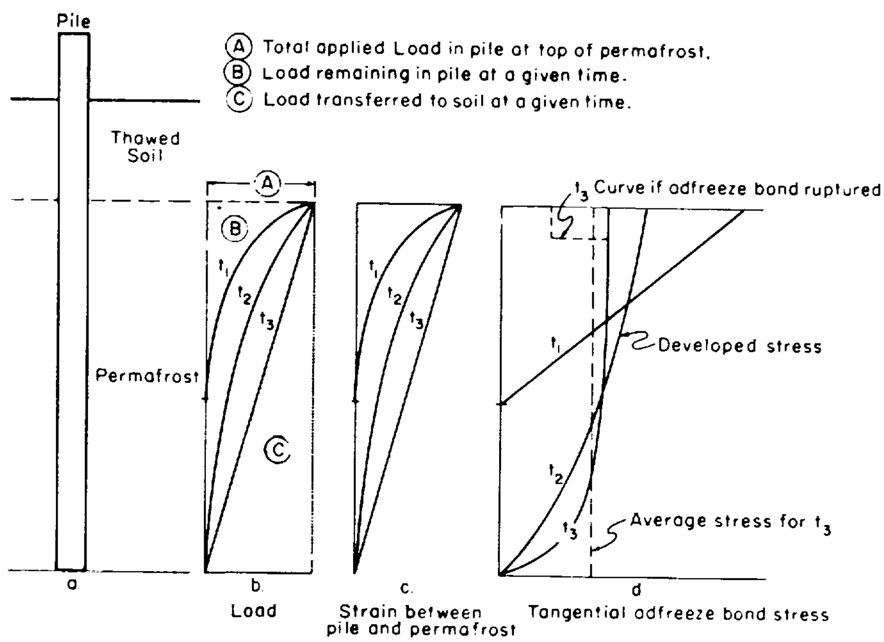

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Figure 4-79. Load, strain and stress distribution for pile in permafrost with zero load at tip.

ed by the surrounding frozen ground. As yield occurs in the permafrost and the adfreeze bond, more and more compressive strain develops in the pile with depth, progressively readjusting the pattern of load transfer from pile to soil, the strain at the pile-permafrost interface, and the tangential adfreeze bond stress. As shown in diagrams b. and c. in figure 4-79, the final distributions of load and strain along the embedded length of pile in permafrost at time t, are approximately triangular. An assumed pattern of adfreeze bond stress is shown in diagram d. of figure 4-79. As also shown in this diagram, the adfreeze bond may be ruptured as a result of excessive stress or excessive rate or magnitude of strain, beginning at the top of permafrost. The stress strain-time relationship and possibility for bond rupture are affected not only by the behavior characteristics of the frozen soil and the adfreeze bond zone, but also by the deformation characteristics of the pile. Piles which exhibit high deformation per unit length under load or which are especially long are more susceptible to such bond rupture.

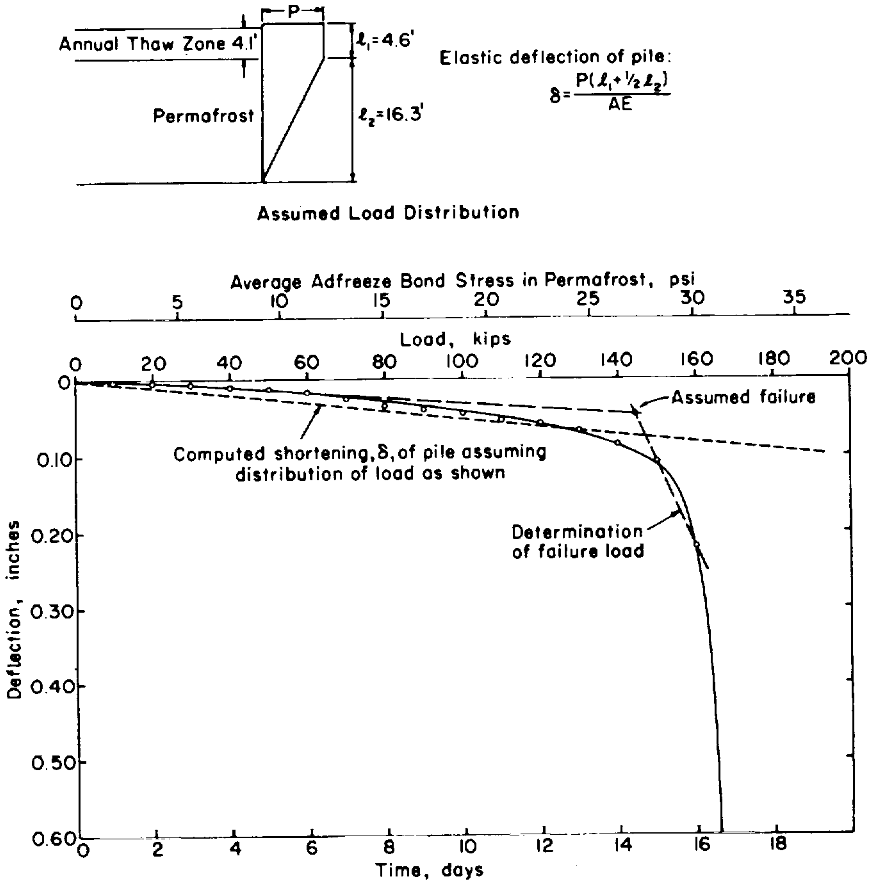
(d) That a triangular distribution of load and strain over the depth of embedment, reducing to zero load at the tip, may be reasonably assumed as a basis for design in relatively warm permafrost is demonstrated in figure 4-80, which shows load-deflection data from a compression load test to failure on an 8-inch pipe pile compared with deflection computed on the basis of the triangular load distribution assumption. The degree of correspondence appears especially satisfactory when account is taken of the fact that application of factor of safety in the design will place working loads in the area of best agreement. It must be recognized that even though the load was added slowly over more than two weeks, in the test shown in figure 4-80, the stress-strain adjustment under each 10-kip increment was not 100 percent complete. In the test illustrated in figure 4-81, measured distributions of strain in an 8-inch. I-beam pile with 10-feet embedment in permafrost under various levels of imposed loading also show the general triangular pattern[83].

(e) For friction type piles of average lengths, bearing on the tip is small enough so that it can be ignored if the tip diameter is relatively small (of the order of 6 inches) or if the pile is placed in a dry-augered hole which is not flat-bottomed and/or if loose auger cuttings are unavoidably left at the bottom of the hole, thus requiring appreciable strain at the tip before full end bearing can be achieved. If the tip diameter is relatively large and if full positive end contact is assured, results obtained by ignoring the load on the tip may be too conservative. In such case, the pile tip load may be computed as the sum of the first two factors in the applicable equation of figure 4-61b.

(f) Effective unit values of the strength of adfreeze bond between frozen soils and foundation piles under long term loading depend primarily on such factors as the type of soil, the moisture content, the chemical composition of the pore water, the temperature, and the surface condition, shape and length of the pile. With augered and slurried piles some control can be effected over the adfreeze bond strength that can be developed, by controlling the type of pile material and surface, the soil type and moisture content of the slurry, the mode of freezing, and the characteristics of the water used to make the slurry. With driven steel piles such control is not possible except for removal of oil, paint, rust or scale from the pile surface before driving; however, there is no freezeback delay or uncertainty, a common problem with slurried piles.

(g) Experimentally determined values of average sustained and average peak adfreeze bond strengths for frozen slurries made with silt of low organic content in contact with steel pipe piles of 18 to 21-feet lengths (in frozen soil) are shown in figure 4-82. Factors for adjusting the curves for different types of piling and slurry backfill, based on field and laboratory testing, are also shown in the figure. (The curve "Average Sustainable Adfreeze Strength" with the appropriate correction factor for variation in pile and/or slurry type and with a procedure to be illustrated later may be used for preliminary design and planning of pile load tests.) Because shear strain along the surface of a loaded pile varies along the length, decreasing downward from a maximum at the ground surface, as has been illustrated in figure 4-79, such values measured on full scale piles represent averages over wide ranges of development of the stress-strain curve. The average values are therefore always less than the potential maximum adfreeze bond stress. Average adfreeze bond strengths at ultimate pile bearing capacity are about 40 percent greater than average sustainable adfreeze bond strengths used in design (before application of any factor of safety).

(h) Adfreeze bond strengths and creep properties of slurry may range from those characteristic of freshwater ice, through those of frozen sands, silts, clays and organic soils at various moisture contents (depending on the type of material selected, the water content at freezeback, and the manner of freezing) to those of the same soils unfrozen, if freezeback is incomplete or if permafrost degradation should occur. In temperature ranges a few degrees below 32 °F, slurries in which the ice fraction predominates may show better structural performance than slurries of some soils in which the soil fraction is more predominant, if the solute content of the added water is relatively low, depending on the soil type. As shown in figures 2-12 and 2-13, ice has relatively high ultimate strength compared to most frozen soils at temperatures immediately



Pile type: 8-in. pipe, 36 Ib/ft
Pile length: 20.9 ft
Length below surface: 20.4 ft
Embedment in frozen soil: 16.1 ft'

Soil profile: 0-1 ft peat, 1-20,4 ft (bottom of pile) silt
Backfill around pile: silt-water slurry
Avg temp of frozen soil: 29.2°F
Test performed: July 1958

Loading schedule: 10-kip increments applied at 24-hr intervals. The deflection shown for an increment is that observed just prior to application of next increment.

Note: Pile not isolated from soil in thaw zone.

COMPUTATION OF ALLOWABLE DESIGN LOAD

Failure load = 147 kipsSurface area of pile in permafrost = 5230 in.^{2}

Average adfreeze bond stress at failure = 147 kips/5230 in.^{2} = 28.1 psi

Adjusting for 10 kips per day rate of loading (by interpolation, Figure 4-85), average adfreeze bond stress at failure = 21.5 psi

Assuming failure stress is 40% greater than average sustainable stress, average sustainable adfreeze strength = 21.5/1.4 = 15.3 psi

Sustainable pile load capacity = 15.3 psi x 5230 in.2 = 80 kips

Using a factor of safety = 2.5, allowable design load = adjusted failure load/FS = 21.5 x 5230/2.5 = 45,000.1b

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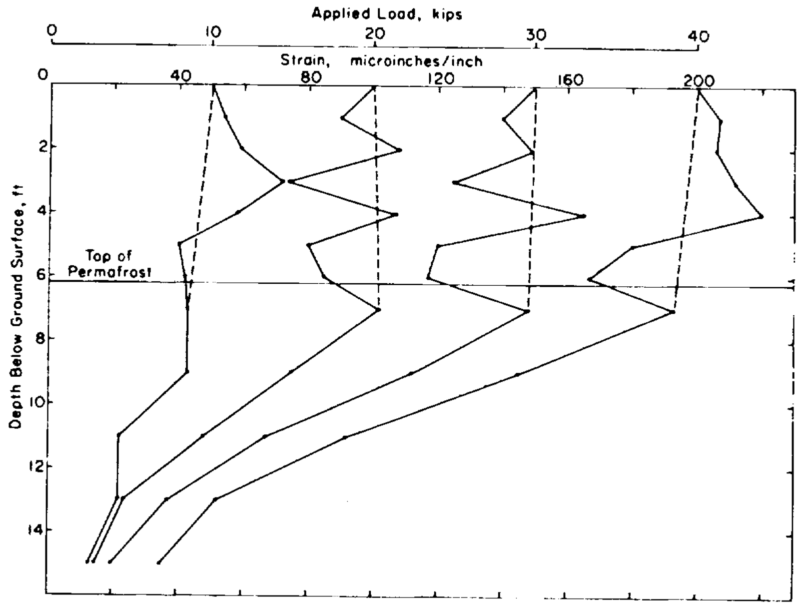
Figure 4-80. Load test of steel pipe pile.



Pile type 8-in. I-beam, 18.4 lb/ft
(bottom of pile) silt
Pile length 16.8 ft
Length below surface. 16.1 ft
Embedment in frozen soil 9 9 ft
Soil profile, 0-1 ft peat, 1-16.1 ft
Backfill around pile Silt-water slurry
Avg temp of frozen soil. 30.8°F
Test performed December 1961
Loading schedule 5-kip increments applied at 30-minute intervals to total load of

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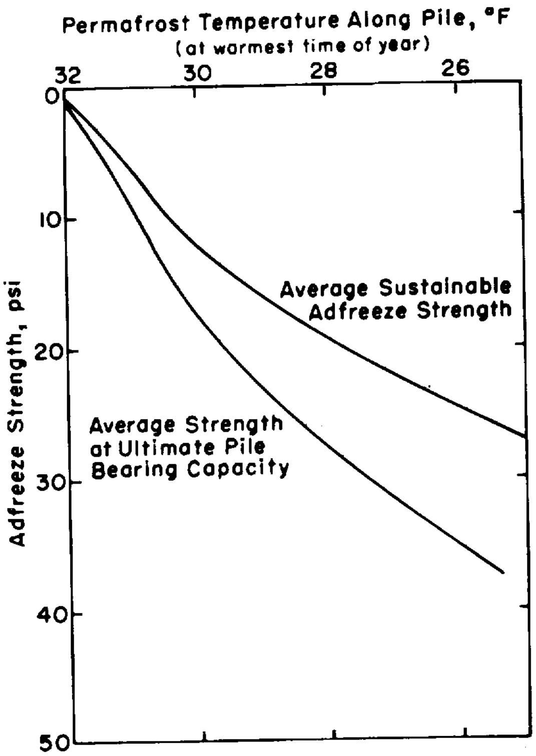
Figure 4-81. Load distribution along pile during test, strain-gage instrumented pile. Frost penetration of about 1 foot occurred adjacent to pile prior to test. Adfreeze bond of frost broken at surface and use of heating devices on ground surface prevented additional frost penetration.



Correction factors for type of pile and slurry backfill (using steel in slurry of low-organic silt as 1.0)

	Slurry soil	
Type of pile	Silt	Sand
Steel	1.0	1.5
Concrete	1.5	1.5
Wood, untreated or lightly creosoted	1.5	1.5
Wood, medium creosoted (no surface film)	1.0	1.5
Wood, coal tar-treated (heavily coated)	0.8	0.8

Notes:

1. Applies only for soil temperatures down to about 25° F.
2. Where factor is the same for silt and sand, the surface coating on the pile controls, regardless of type of slurry. In the remaining factors the pile is capable of generating sufficient bond so that the slurry material controls.
3. Gradations typical of soils used for slurry backfill are shown in Figure 2-11 as follows:
   Silt - SFS, Fairbanks silt
   Sand - SM, McNamara concrete sand
4. Pile load tests performed using 10 kips/day load increment were adjusted to 10 kips/3 day increment to obtain curves shown.
5. Clays and highly organic soils should be expected to have lower adfreeze bond strengths.

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Figure 4-82. Tangential adfreeze bond strengths vs. temperature for silt-water slurried 8.625-inches-O.D. steel pipe piles in permafrost averaged over 18 to 21 feet embedded lengths in permafrost[134].

below freezing when load is increased relatively rapidly to failure, but most frozen soils exceed the strength of ice at lower temperatures. At a temperature of about 30 °F, freshwater ice, frozen concrete sand and fine sand have shear strengths of about the same magnitude, but frozen silt is significantly weaker. With lowering of temperature, ice does not gain further shear strength, but the frozen soils do. At temperatures between 30° and 25 °F, shear strength of sands may exceed that of silt by 33 percent to more than 100 percent. At a temperature of about 20°F sands and silt may have about equal shear strengths, but these may exceed the shear strength of ice substantially. As temperatures fall below 200°F, silt continues to increase in shear strength at a rate which is much more rapid than for sand. In absence of reliable direct adfreeze bond strength data, shear strength behavior is considered the most nearly analogous characteristic. Also, as illustrated by figure 2-15, hard, sound freshwater ice shows a lower rate of creep deformation than frozen soils, at least in the temperature range above about 26 F; data are not available for lower temperatures, or for lower temperatures, or for ice which is porous or contains significant amounts of impurities. However, the much more rapid rates of freezeback obtainable with minimum moisture content slurries offer a significant construction advantage[134]. Concrete sand also will usually contain few soluble materials to alter freezing temperature of the pore water.

(i) The use of steel H-type piles or other irregular section piles, which have considerably more surface area than a comparable-size circular pile, will not result in proportionately greater pile load capacity simply because of this increased surface. Although extraction tests to failure of H-piles show that such piles come out clean, i.e., without soil included between the flanges, the included perimeter should be used in design rather than the actual surface, since yield in creep will tend to occur on that basis.

(1) Available data indicate that when steel piles are driven into permafrost by conventional methods, adfreeze bond over the pile surface area is less complete than is possible with a properly placed slurry backfill. This conclusion is based both upon load tests and inspection, for evidences of contact, of piles which have been completely extracted. Therefore, the allowable load-bearing capacity of a conventionally driven steel pipe pile should be reduced to 75 percent of that for a slurried pile in which the slurry is made from the same foundation soil mixed with fresh water. For H-type and other irregular section piles, the reduction should be less since the pile surface area allowable is partly direct pile soil contact surface and partly surface through frozen soil as computed for the included perimeter. For example, the allowable load bearing capacity for a driven 10 inches by 10 inches steel H-pile would be 87.5 percent of that for a slurried pile, if the sustainable creep strength of the soil is approximately the same as the unreduced adfreeze strength between soil and steel. However, if a pile is driven by a method which generates enough heat to produce a slurry film on the entire pile surface in permafrost, no reduction is needed.

(k) The possibility must be considered that the natural foundation soil may have sufficiently low shear strength, as compared with the adfreeze bond strength at the slurry/pile interface, to be the controlling factor in determining the load-carrying capacity. However, since the perimeter of the drilled or augered pile hole is ordinarily a minimum of 30 to 50 percent greater than the perimeter of the pile itself, the natural frozen ground would have to be much weaker than the strength at the slurry/pile interface for the strength of the natural soil to be controlling. This is very unlikely if the slurry is made from the natural foundation soil. However, it may occur if select slurry backfill is used. It is possible to intentionally make the augered hole larger than needed for pile placement purposes alone, in order to decrease stress at the outside perimeter of the slurry cylinder when this condition may apply. However, the resulting increase in slurry volume would significantly retard freezeback time.

(1) For preliminary design purposes, the average sustainable adfreeze strength values in figure 4-82, adjusted by the appropriate factor if necessary, should be used. Normal negative frictional resistance values (combined friction and cohesion) for unfrozen soil may be determined from guidance given in TM 5-818-1/AFM88-3, Chapter 7. Full-scale load tests performed on piles installed by the planned construction procedures best integrate the variables involved. Figure 4-80 shows an example of such a test, though the interval between load increments is less than the 10 kips per three days which is recommended (see below). However, if sufficient time is unavailable in the construction schedule, such deviation may sometimes have to be accepted and corrected for as indicated in the "Computation of Allowable Design Load" in figure 4-80. If these tests cannot be performed at the design ground temperature (as is frequently the case for field tests), they may be adjusted to the design temperature by using the applicable curve in figure 4-82 for guidance and assuming that the strength for the test case varies as a fixed percentage of this curve with temperature.

(m) Since the effective unit adfreeze bond strengths are directly related to permafrost temperatures, reasonably accurate assessment of the permafrost temperatures with depth, for the life of the structure, is required. The warmest temperatures with depth to be experienced in the life of the structure should be used for design. For ventilated pile foundations this normally will occur in the early part of each winter. If a residual thaw zone should exist or develop, there will be no seasonal variation in permafrost temperatures and the permafrost will tend to eventually reach a thawing condition'"; in such a case the recommended procedure is to design for thawed-condition skin friction values. If the permafrost or slurry contains excess moisture in the form of ice, these values will tend to be on the low side for the type of soil involved, and negative friction forces may need to be considered. However, since thaw at depth in the ground is usually slow, consolidation will normally occur in small annual increments.

(n) If permafrost temperatures in the seasonal period selected for design are essentially uniform with depth, the permafrost supporting capacity may be estimated (using fig. 4-82) by simply multiplying the total surface area of the part of the pile in permafrost by the average sustainable tangential adfreeze bond strength at that temperature, applying a correction factor for the type of pile surface and slurry if necessary. Should the ground temperatures vary appreciably with depth, a more refined computation of the permafrost supporting capacity may be made by plotting, first the variation of average sustainable bond stress with temperature, then temperature with depth, then the average sustainable bond stress for the applicable temperature with depth, and finally the sustainable adfreeze bond load capacity per foot with depth. By determining the area under the latter curve as shown in the right hand diagram of figure 4-83, the potential pile load capacity is obtained.

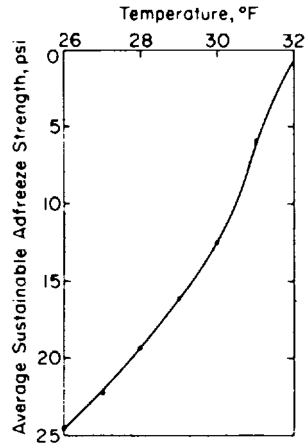

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Figure 4-83a. Example of computation of sustainable load capacity of pile in permafrost. (Average sustainable and adfreeze strength vs. temperature. Values taken from figure 4-82. If an estimate is desired of the load capacity of the pile at failure, values from the curve "Average strength at Ultimate Pile Bearing Capacity " in figure 4-82 should be used.)

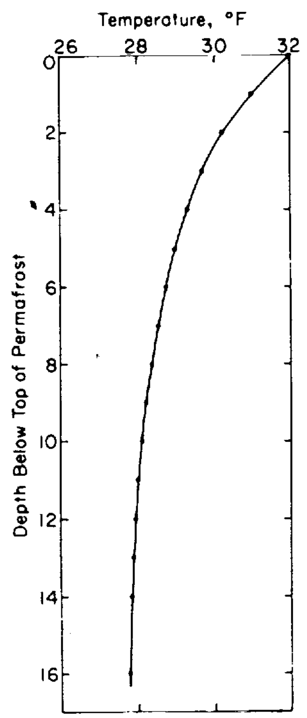

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Figure 4-83b. Example of computation of sustainable load capacity of pile in permafrost. (Temperature vs. depth below top of permafrost, warmest time of year.)

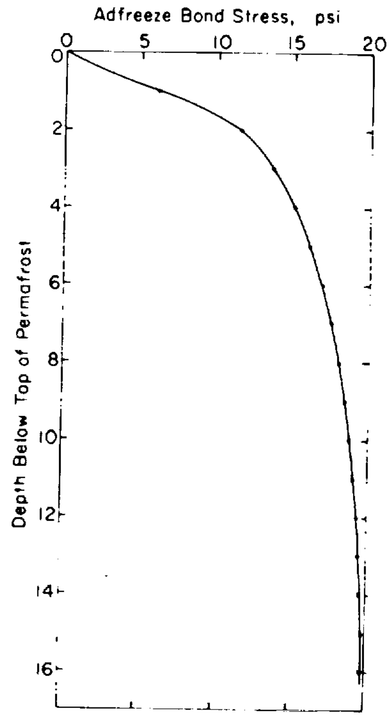

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Figure 4-83c. Example of computation of sustainable load capacity of pile in permafrost. (Average sustainable adfreeze strength for applicable temperature at depth below top of permafrost.)

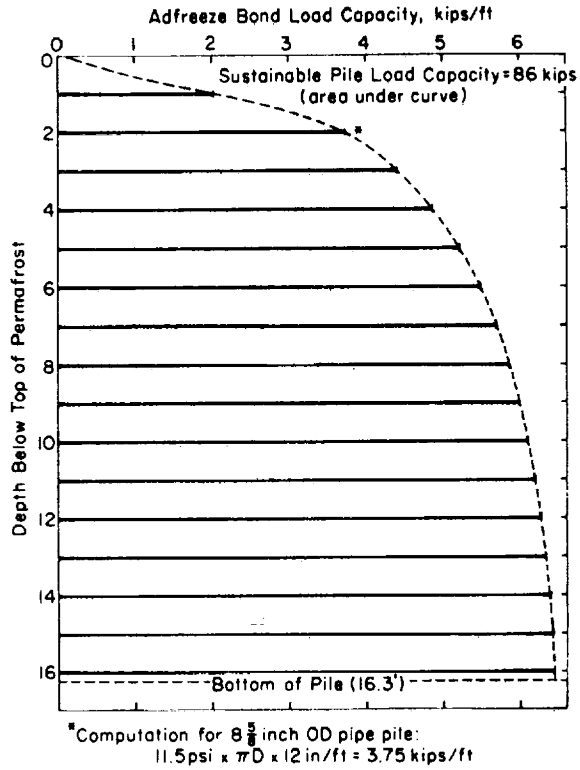


Figure 4-83d. Example of computation of sustainable load capacity of pile in permafrost. (Adfreeze bond load capacity vs. depth below top of permafrost

(o) Within reasonable limits a deflection of the pile relative to the surrounding permafrost, which exceeds the minimum strain required to develop peak adfreeze bond stress at the top of permafrost, is normal and acceptable in permafrost at a temperature of about 20° to 25 °F or warmer, provided opportunity for gradual development of this displacement by creep is available. The curves for slow loading shown in figure 4-78 typify this condition. However, the possibility of complete rupture of the adfreeze bond in upper permafrost strata must be considered and special analysis should be made when considering piles of significantly more than 30 foot embedment in permafrost, or if the temperature of the permafrost when loads are applied is colder than about 20 °F, or full design load is suddenly applied on the pile. The criteria in this manual are based on experience with piles of conventional lengths of permafrost embedment, that is 15 to 30 feet in permafrost at temperatures of 20° to 25°F or warmer, under gradual application of load.

(p) In permafrost of low and very low temperatures (colder than about 20°F) unit adfreeze bond strengths are higher, allowable deflections are lower, optimum pile lengths are less, possibility for rupture of the adfreeze bond is increased, and the patterns of distribution of load, strain, and stress along the embedded length of pile may differ from the pattern which has been described above because the stress-strain behavior typified by the two left-hand curves in figure 4-78 will apply rather than the extended-strain type behavior shown in the two right-hand curves of that figure. Even though higher stress levels can be accepted under rapid loading, reduced capacity for readjustment by creep may nullify this.

(q) The computation of the allowable load on the pile should be completed using equation 14 above and factor of safety from h below. TM 5-818-1/AFM 88-3, Chapter 7 may be referred to for guidance concerning skin friction of thawed soil.

(2) End bearing piles. As described in the preceding paragraph, the point bearing (Q_e in fig. 4-77) may often be assumed negligible. However, if a firm, reliable bearing stratum such as ice-free bedrock is within economical depth, the bearing capacity can be augmented by or solely derived from end bearing. Design procedures for end bearing piles should be the same as in temperate zone practice (TM 5-818-1/AFM 88-3, Chap. 7' and/or EM 1110-2-2906[20]) except that safety against frost heave must be assured in accordance with the following paragraph. Drilling and anchoring of

the piles into the bearing stratum may be required.

(3) *Pile safety against frost heave*.

(a) Analysis must be made to assure that the pile is safe against frost heaving under the normal sustained dead load, or when not loaded if this can occur during a freezing period. The latter is most likely to happen during construction.

(b) For heave stability a satisfactory relationship as expressed in the following equation must be maintained under the most unfavorable conditions (see lefthand side of fig. 4-77):

$$Q_h = \frac{1}{FS}(Q_L + Q_p + Q_t) \qquad \text{(Equation 15)}$$

where

$$Q_h = \text{frost heave force} = \int_{\ell_4}^{\ell_5} f_h dA_n$$

where

A_n = surface area of pile in seasonally frozen ground

f_h = adfreeze bond stress mobilized between frozen soil and pile by heave

Q_L = effective load of structure, P, and pile, W

Q_p = same as in equation 14 above, acting from l_6 to f $_7$, but stress mobilized in opposite direction

Qt = skin friction of thawed soil on pile

$$= \int_{\ell_5}^{\ell_6} f_s dA_t$$

If the pile can experience loading in tension from the structure the equation must be adjusted accordingly. The nature and mechanism of the heave phenomenon and values of frost uplift pressures which can act at the plane of freezing are discussed in paragraphs 2-2 and 2-4. The effects of frost heave on engineering structures and methods of controlling these effects are discussed in paragraph 4-3. Because a pile in tension contracts in the transverse direction, there is a tendency for skin friction under high tensile load to be less than in compression. As shown in figures 4-44 and 4-45, peak frost heave forces have been measured of 55,800 lb or 2220 lb per perimeter-inch for an 8 inch steel pipe pile and 35,60 lb or 809 lb per perimeter-inch for a creosoted timber pile, both in frozen silt slurry. For steel piles in silt with ample moisture available, an average value of fh of 40 psi should be assumed to act over the full depth of seasonal freezing; in the coldest upper strata of seasonally frozen soil, local tangential shear stresses on the surface of the pile may be substantially higher. For other pile and soil types the 40 psi value may be adjusted proportionately to factors noted in figure 4-82. Experience shows that the surface of the pile within the annual thaw zone will often show a thin coating of ice which may possibly control the observed behavior of the system. Although clays are capable of producing considerably higher heaving pressures than silt (see fig. 2-9), values of fh for clays should be less than for silt because of the limited heave rates possible with these low permeability soils and because of their weaker adfreeze bond potentials.

(c) The holding force in permafrost, Qp, should be computed as in (I) above using average sustainable adfreeze strength. A factor of safety must be applied as outlined in paragraph h below.

(d) If the capacity of the pile is insufficient to resist the frost thrust, Q_h, the pile must be redesigned to provide greater loading, increased embedment or holding power in permafrost, or a combination of these alternatives, or one of the heave force isolation methods discussed in paragraph 4-3 may be restored to. Where frost heave is possible, wood piles should be installed butt down to increase safety against heave. Piles which heave as the result of frost action destroy the adhesive bond of permafrost. Once broken this bond does not readily reheal and as little as 1/2 or less of the potential adfreeze bond strength may be available to support the imposed load and to resist further heaving in subsequent years.

(4) Tension loading. It may sometimes be desirable to use piles in frozen ground to resist tension or uplift loads. Their advantage usually is their low cost. However, the use of friction piles in frozen ground to carry permanent tension loads should be approached with great caution for the following reasons: friction piles in frozen ground have inherent potential to fail progressively in creep; piles in tension tend to experience reduction in friction by transverse contraction under load; frost heave forces act in the same direction as the applied stress; permafrost degradation or even warming of ground temperatures during the life of the structure may lead to failure; developing failure under a structure, if observed, may be very difficult to correct; and failure, if it occurs, may be accelerative and catastrophic. Some of these adverse factors can be eliminated by careful design. For example by means of a rod passing through the pile it can be placed in compression rather than tension; in such case it may be better designated as an anchorage than as a pile. Ample factors of safety can also be employed. However, in lieu of friction piles positive gravity or mechanical type anchorages which insure mobilization of the required mass of soil and are less sensitive to design, construction and

operational deficiencies and uncertainties can be used. For short-term, intermittent tension loading, as from wind, the problem is less critical; in this case the pile may be designed by adaptation of the procedure outlined in the previous paragraph for safety against frost heave. Again, however, the designer must make sure that there is no possibility of unacceptable degradation or warming of the ground during the life of the facility, either from natural conditions or from improper practices on the part of the facility operators, and a very conservative approach must be taken. The decision may rest in considerable part upon how serious would be the probable consequences of a failure.

g. Load testing of piles in permafrost.

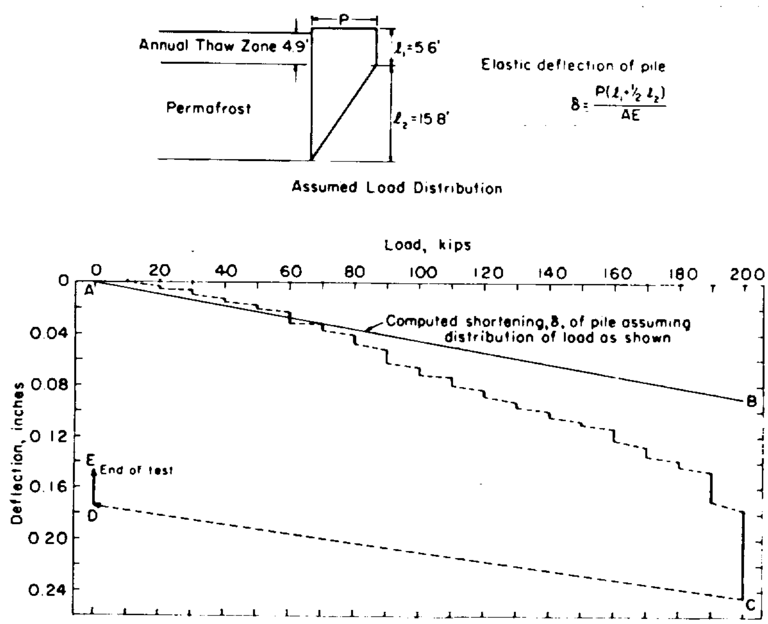
(1) Pile testing in permafrost may be required or desirable to obtain data needed for design, to verify design assumptions, and/or to evaluate various alternative designs. In addition to more or less conventional load settlement and extraction pile tests, cyclic, long term static load or lateral load tests may be needed under some conditions[134,168]. In addition to the direct load capacity value of the pile test data, considerable useful collateral information can be derived in the course of performing a pile test program. Such information may cover rates and times required for freezeback; ease of driving, augering or drilling; techniques and problems of mixing and placing slurry backfill, and supplementary foundation soil information (as from auger cuttings). The pile load tests should normally be performed during the facility design studies. However, they may also be performed at the start of the foundation construction if their function is to verify design assumptions, provided opportunity for design adjustment exists.

(2) While a pile load test set-up may be based on the general methods outlined in ASTM D1143[118], the following special procedures must be followed in testing piles in frozen ground.

(3) All vertical instrumentation supports shall be 2-inches or larger pipes driven 20 or more feet into frozen soil and cased if necessary to isolate them from frost heave. All instrumentation pipes shall be 8 or more feet from test and anchor piles or load platform supports. In addition to checking observations with an engineer's level, the motion of the test pile under load shall be frequently monitored by dial gages having 0.001 inch subdivisions and having 2 or more inches of travel. At least three such dials shall be used on round piles, at 120° intervals, and four dials on other piles. All dials shall be equally spaced and equi-distant from the pile center, on a common horizontal plane. The dial gage support beam shall be roller-supported at one end to avoid bending of the beam as a result of thermal expansion and contraction forces. During the loading and unloading of the pile the dials shall be observed at sufficient time intervals to permit the plotting of accurate settlement vs. time and settlement vs. load data to 0.001 inch. The dial gages and their supports shall be completely protected from direct sunlight, and from precipitation and wind, by a suitable shelter. The shelter must be ventable to minimize the build-up of heat during the day. As nearly uniform temperature as practicable should be sought within the shelter. Air temperatures within the shelter shall be observed at least hourly at the instrumentation level. Ground temperatures with depth at test piles shall be monitored daily to establish the rate of freezeback following installation and the temperature conditions in the surrounding ground during the load testing period. In winter, local heating shall be used as necessary to avoid the possibility of frost formation on instruments and consequent malfunction (ordinary electric light bulbs are often the most convenient source of such heat). All load increments shall be added with care to avoid producing impact overload.

(4) For a site where little or no previous pile bearing capacity information is available, a minimum of one exploratory pile load test and one verification pile load test (see below) should be performed during design investigations. Only a verification pile load test during design or at start of construction may be necessary if previous pile bearing capacity information is available for the construction site or the soil formation, or if the job is small and a conservative value has been assumed for design load in lieu of making detailed pile bearing studies. Verification pile load tests should be made in the construction stage on all major projects.

(5) In exploratory pile load tests the load is increased progressively in relatively small increments in order to define changes of a pile response with load with reasonable accuracy. To obtain an estimate of pile load capacity, figure 4-82 should be used along with the procedure illustrated in steps from left to right in figure 4-83. A standard increment of 10 kips is assumed in the following discussion. Loading should be continued to failure, normally defined to occur when the gross settlement reaches 1.5 inch, or to 2 /z times the anticipated design load. One example of such a load test, in which a loading rate of 10 kips per day was employed, has been shown in figure 4-79. Figure 4-84 shows results of another test in which load increments were added at the much slower rate of 10 kips every 4 days, except for a number of loads which were held for longer periods, up to 12 days. Figure 4-84 has been prepared with an expanded vertical scale in order to show the deflections which occurred under the individual load increments. The cumulative deflection curve AC shows about 0.16 inch greater deflection at the maximum load than that (at point B) computed from the assumed load distribution pattern, shown on the figure. This difference cor-



Pile type: 10BP42

Pile length 21.4 ft
Length of pile below surface: 20.7 ft
Pile embedment in frozen soil 15.8 ft
Pile cased through active zone.

Soil profile 0-1 ft peat, 1-20 7 ft
(bottom of piel silt
Backfill around pile silt-water slurry
Avg temp of frozen soil 28°F
Test performed Feb-May 1961

Loading schedule: 10-kip increments at 4-day intervals except load held at 90 kips for 8 days, at 130 kips for 6 days, at 160 kips for 8 days, at 170 kips for 5 days, at 190 kips for 12 days, and at 200 kips.for 9 days. Test terminated due to failure of hydraulic jack and zero load rebound observed for 3 days.

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Figure 4-84. Load-settlement test. 1O-kip increments.

responds approximately with the permafrost displacement remaining at point E after removal of the load, C to D, and completion of rebound. At a practical working load of about 72 kips, obtained by dividing the ultimate load (as defined at the end of this paragraph) of about 180 kips by a factor of safety of 2.5, relatively good correspondence may be observed between the computed elastic deflection (curve AB) and the observed deflection (AC).

(6) To provide meaningful test data, the false capacities achieved by rapid rates of loading should be avoided by limiting the rate of load increase. The required time interval between load increments increases with increase in length of pile embedment, decrease of permafrost temperature, and increase in relative intensity of loading. For piles of less than about 20 feet embedment, in permafrost warmer than about 24 F load increments should be maintained for at least 72 hours. It is simplest to use a uniform period between additions of load increments throughout each test. For longer piles or lower permafrost temperatures, it may be necessary to use longer time intervals; this must be determined by test. Figure 4-85 shows the effect of time between load increments on tangential adfreeze bond failure stress on 6-inch steel pipe piles of 11 to 12 feet embedment in permafrost installed in augered holes backfilled with siltwater slurry, tested at permafrost temperatures down to 23.5 'F. It will be apparent that for 10,000-lb increments held less than about 72 hours, measured bond strength values for these piles tend to be too high, requiring application for correction factors obtained from these curves to reduce the results to long term values. Similar relationships may be assumed to apply for other types of piles. Thus, the observed ultimate adfreeze bond strength obtained in the load test on a steel pile shown in figure 4-81 is too high because the 10-kip increments

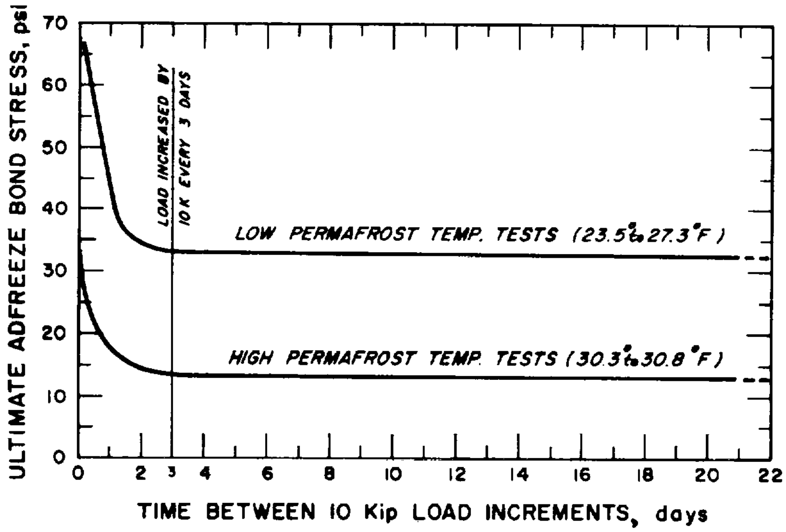

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Figure 4-85. Effect of rate of loading and temperature on adfreeze strength of steel pipe piles[134]. Curves based on tests of 6 inch steel pipe piles installed in augered holes backfilled with silt-water slurry at Fairbanks, Alaska. Embedment of piles in permafrost varied from 10.9 to 12.0 ft.

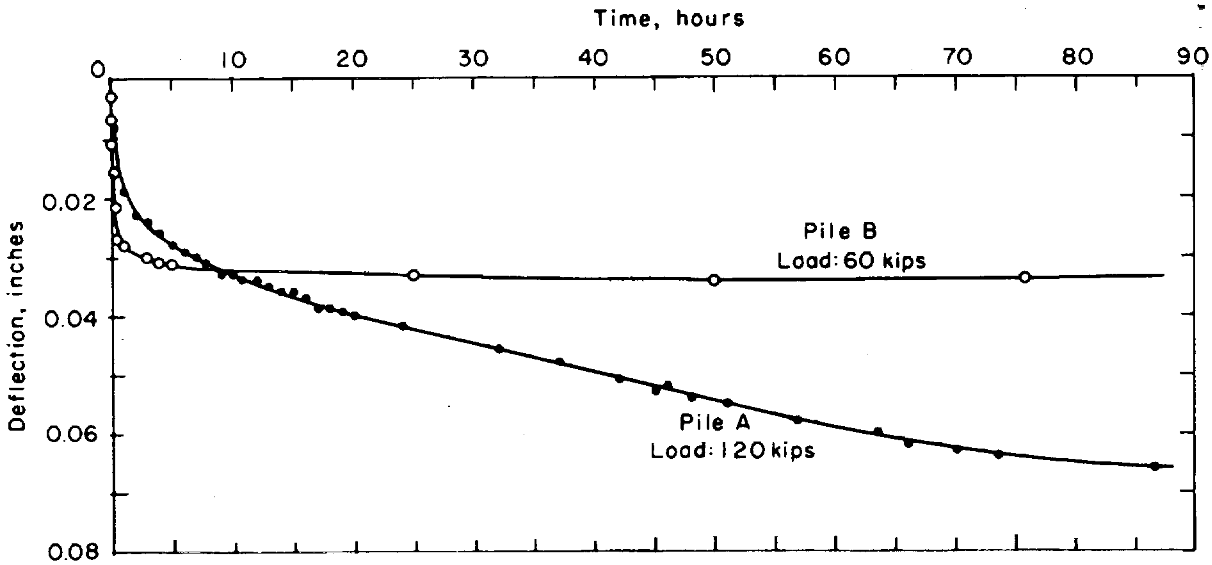
were only held one day. However, that obtained in the test shown in figure 4-84 with each increment held at least four days requires no correction. A continuous record of deformations should be obtained under each load increment and continuing deformation rates at ends of the increment periods should be plotted against load to assist in determination of the load level at which excessive creep deformations begin. However, it should be kept in mind that stress redistribution along the length of the pile under an increment of load may continue far beyond a period of even three days and that such readings should not be assumed to quantitatively represent creep rates under long term steady load. The load may be completely removed at intervals during the test and the rebound of the pile noted. The rebound of the pile after the maximum load had been released should be observed for at least 24 hours. The deformation of the pile after rebound (point E in fig. 4-84) is known as the net or plastic deformation. The algebraic difference between the total deformation and the net settlement (difference in deflection between C and E in fig. 4-84) is known as the elastic deformation of the pile and soil. The net or plastic deformation of piles in permafrost rarely exceeds 0.50 inch before complete failure of the pile.

(7) As shown in figure 4-84, analysis is aided by comparing the observed deformations with the computed pile shortening. Comparison may also be made with equivalent end bearing piles. Such analyses give indications of the length of pile actively supporting load and assist in recognition of failure situations.

(8) A value of failure or ultimate load should be determined from the load test results. A number of common criteria for selection of failure load are listed in TM 5-818-1/AFM 88-3, Chapter 7[5]. The most appropriate of these for tests in frozen ground is that which defines the failure or ultimate load as the load indicated by intersection of tangent lines drawn through the initial, flatter portion of the load-deformation curve and through the steeper part of the same curve. Adjustment to the critical design temperature should then be made if required, using the data shown in figure 4-82 and the allowable design load should be computed by application of a factor of safety as indicated in h below.

(9) If the length of time required to perform an exploratory load test as described is unacceptable, an alternative approach is to perform simultaneously several verification pile load tests as described below, with load values selected so as to positively bracket and establish the acceptable design load.

(10) *In verification pile load* tests the pile may be loaded to the design load in a single increment and then to 2 1/2 times the design load in a second increment, all other requirements remaining the same as for an ex-



	Pile A	Pile B
Pile type:	Wood, butt down	Wood, tip down
Avg pile diameter:	11 in.	12 in.
Pile length:	34.5 ft	21.4 ft
Length of pile below surface:	31.0 ft	21.0 ft
Pile embedment in frozen soil:	31.0 ft	16.5 ft
Backfill around pile:	Silt-water slurry	Silt-water slurry
Soil profile:	0-1.6 ft gravel fill 1.6-3.3 ft peat 3.3-15 ft silt 15-31 ft ice	0.0-1.0 ft peat 1.0-21 ft silt
Avg temp of frozen soil:	24°F	30.4°F
Test performed:	Oct 1955	Nov-Dec 1958

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Figure 4-86. Load settlement test. single increment[37].

ploratory load test. Two examples of deflection measured under single-increment loads are shown in figure 4-86. The much longer stress adjustment time required for the longer pile A, loaded to 120 kips and embedded in 24 F permafrost, as compared to the shorter pile B, loaded to 60 kips and embedded in 30.4°F permafrost, is readily apparent. If significant continuing deflection is still occurring 72 hours after application of the design load increment, observations should be continued until a firm conclusion can be drawn as to whether or not the pile will be safe against excessive creep deformation under the design load. It is not necessary for the rate of deflection to drop entirely to zero. Normally proof of safe bearing capacity will not be a problem because the design load, which includes a factor of safety, will be conservatively low. Application of the second increment, increasing the load to 2 ½ times the design load, is intended primarily to provide a further check on the validity of the design load. Deflection measurements normally need not extend beyond 72 hours for this second increment, regardless of rate of continuing deflection. Intermediate increments of load between these two may be used if time and other constraints permit.

h. Factors of safety.

(1) On the basis of failure load determination from pile loading tests (g(5) above) the factor of safety of friction type piles against ultimate failure should be at least 2.5 for dead load plus normal live load and 2.0 for dead load plust maximum live load. Since ultimate adfreeze bond strength is about 1.4 times the sustainable, the factor of safety of 2.5 provides a factor of safety of about 2.5 divided by 1.4 equal to 1.79 with respect to the sustainable strength, on a gross basis. When the allowable design load in equation 14 is computed analytically (f(1) above) the gross factor of safety against ultimate failure contained in the resultant value Q_a should not be less than 3.0. These criteria apply for piles of average length of embedment in permafrost, i.e., 15 to 35 feet.

(2) Factors of safety for end bearing piles should be the same as in TM 5-818-1/AFM 88-3, Chapter 75.

(3) Because, as shown in figures 4-44 and 4-45, peak frost heave forces act for only a fraction of the year, avoidance of rupture of the adfreeze bond in permafrost under peak stresses is a more critical problem in considering pile safety against heave than is progressive upward movement under stresses of creep levels. The same is true for piles subject to intermittent external tension loads. If rupture of the bond occurs, major upward displacement may be expected, which, as noted in f(3) above, is not likely to stabilize. These types of loading are also less predictable in magnitude than downward compression type loadings. Under intermittent tension or frost heave loading of piles, factors of safety of 2.5 and 3.9 with respect to failure loads determined by tests and by computations, respectively, should be applied in equation 15 to forestall failure. These values should also be used for other types of foundations when critical stressing is in tangential shear of adfreeze bond.

4-9. Grade Beams. Grade beams or similar horizontal structural members placed at or just below ground level, which may be subject to uplift, should be avoided when frost-susceptible soils are involved. Instead, full foundation type walls should be substituted. In theory, an alternate procedure is to replace the soil at the grade-beam location for the full frost depth with non-frost-susceptible material for sufficient width so that the non-heaving soil under the beam will not be carried up by heave of the adjacent soil. However, no rational procedure for determining the required width of non-frost-susceptible material is yet available.

4-10. Walls and retaining structures.

a. Bridge abutments, retaining walls, bulkheads and similar structures with unheated foundations are susceptible to frost heave, settlement and overturning forces, when the requisite soil, moisture and freezing conditions are present. They are subject to upward frost thrust acting directly against horizontal foundation surfaces and on vertical surfaces by adfreeze bond of the seasonal frost layer to the structure. They are also subject to lateral thrust from laterally acting frost heave forces; as indicated in figure 4-42, frost heave develops in a direction directly opposite to the direction of frost penetration. If freezing temperatures penetrate through a vertical face, such as a wall, water migration, ice lensing and frost thrust will be oriented in relation to the vertical surface in the same manner as they are to a horizontal surface when frost penetration is downward. The force developed can be sufficient to move or break the wall. Progressive small movements, year after year, can produce substantial permanent tilt, because forces during the thawing period do not act to return the structure toward its original position, as is the case, for example, for pavements. For these reasons, the design of walls and retaining structures requires even more care than the design of conventional footings. The most satisfactory method is to place a backfill of non-frost susceptible material directly behind and adjacent to the wall structure, as shown in figure 4-87a, b, to a thickness equal to the depth of frost penetration, using, if necessary, a 12-inch filter layer next to the finegrained backfill. If differential frost heave would cause a problem on the ground surface at the edge of the nonfrost-susceptible backfill, the latter should be tapered out over sufficient distance to eliminate the problem. Positive drainage of the backfill should be provided; however, the possibility that the drainage system may be blocked by freezing during a significant part of the year must be taken into account in the hydrostatic pressure design assumptions for wall stability analysis.

b. The chart shown in figure 4-88 may be used to estimate the depth of backfill required behind concrete walls in order to confine seasonal freezing to the backfill. For average wall conditions (assuming essentially vertical wall faces) with average exposure to the sun, a surface freezing index equal to 0.9 of the air freezing index should be used. The n-factor of 0.9 is greater than the 0.7 used for pavements kept cleared of snow because of the more positive freedom from the insulating effects of snow and ice, because of 3-dimensional cooling effects associated with a wall and embankment, and because of increased cooling effects of wind. If the wall receives no sunshine during the freezing period, is exposed to substantial wind and remains free of snow or ice, an n-factor of 1.0 should be used. If the wall is located in a southerly latitude, has a southerly exposure and therefore receives much sunshine, the n-factor may be as low as 0.5 to 0.7; however, in very high latitudes, the net radiational heat input may be very small or negative and the n-factor may be 0.7 to 0.9.

The chart may also be used for estimating the depth of frost penetration vertically into granular soil below a

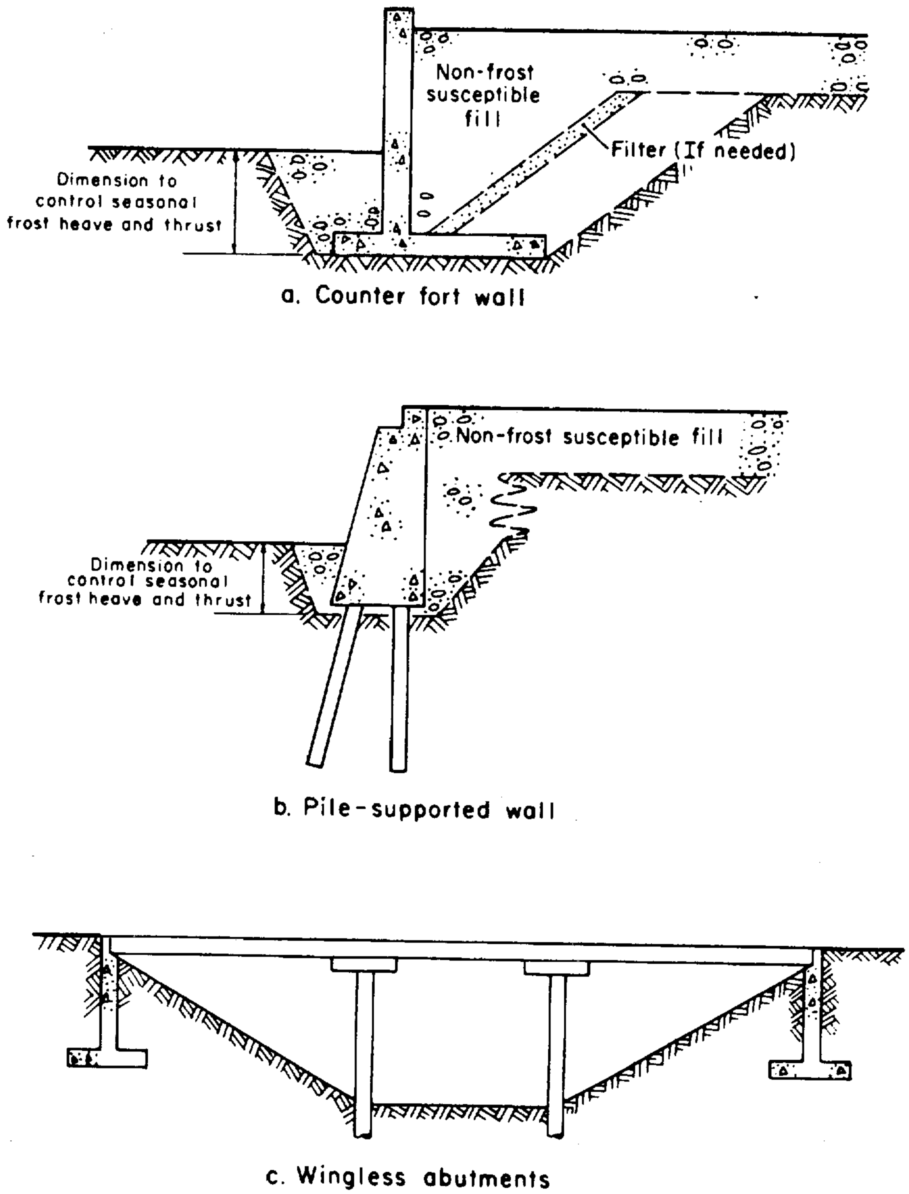

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Figure 4-87. Walls and abutments.

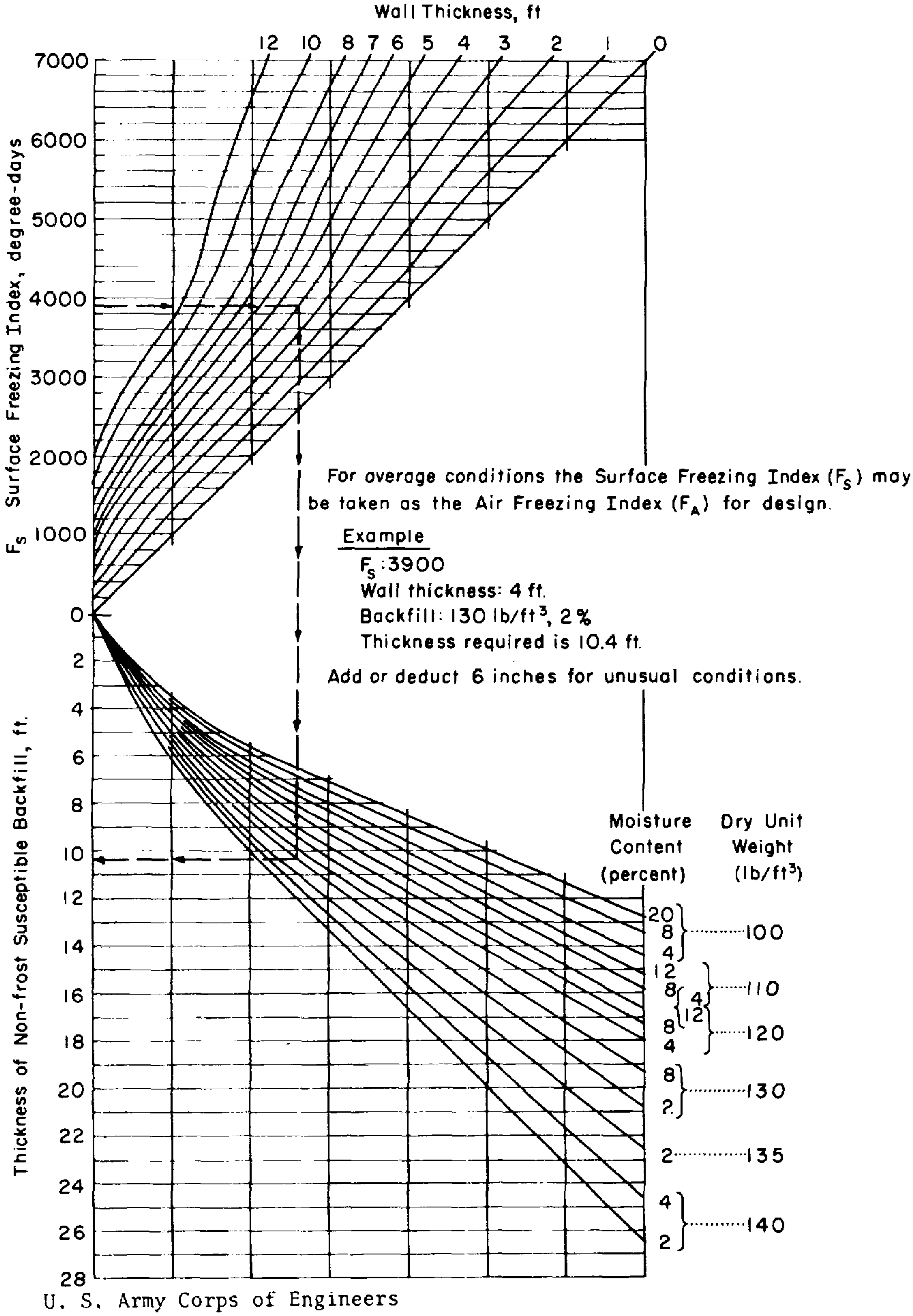

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Figure 4-88. Thickness of non-frost-susceptible backfill behind concrete walls.

snow-free horizontal ground surface; for bare ground, for example, the surface freezing index would be taken as 0.7 of the air freezing index and the chart would be entered with zero wall thickness. If it is necessary for a wall or retaining structure to be in contact with frost susceptible soil over all or part of its height under conditions where freezing direction is vertical rather than lateral, some modification of frost uplift may be provided by battering the face of the structure as much as possible. Heaving soil will then tend to break contact with the wall as it is lifted and thus limit the area of adfreeze contact. It should not be assumed that this will eliminate uplift forces. Anchorage against the uplift forces should be provided by such means as extending the batter well down below the zone of frost penetration.
and/or by using an adequately widespread base. In any case, sufficient reinforcing steel must be incorporated in the concrete to sustain tensile forces developed therein without cracking of the concrete in tension by extension of the reinforcement.

c. In lieu of placing the base of the wall deep enough in the ground so that freezing cannot penetrate under it into frost-susceptible soils, it may sometimes be feasible to support the structure on piles just above the ground, "daylighting" the base sufficiently to provide room for upward expansion of the heaving soil. However, risk is then present that this space may be eliminated by settlement, or by deposition of material within the gap by water or wind, and be unable to function when needed. This is particularly true if the structure is a bridge pier subject to movement and deposition of material by stream flow.

d. Figure 4-87c shows a type of design which minimizes many of the problems inherent in wing or box type bridge abutments. Although it may require special attention to slope stability and erosion control and it requires a longer supported span, it reduces the frost design problems of retaining structures to a minimum and offers much in simplicity.

e. Stability of walls and retaining structures may be computed using earth pressure analytical techniques as presented in TM 5-818-1[5].

4-11. Tower foundations.

a. Towers for transmission lines, communication antennas, cableways or other purposes are commonly either self-supporting as shown schematically in the left hand diagrams of figures 4-89 and 4-90 or guyed as indicated in the right hand diagrams of figure 4-89 and the right hand portions of figure 4-90a,b. When bank-run gravel is available, designs of the types shown in Figure 4-89 may be considered. Figure 4-90 shows a number of possible types of foundations requiring little or no granular non-frost-susceptible material.

b. A tower supported on top of the annual frost zone will experience frost heave if the freezing soil is frostsusceptible and moisture is available. Depending on the design and purpose of the tower, the seasonal vertical movement may or may not be detrimental. If the heave is differential between footings supporting the tower, the tower will tip and/or the structure will be unevenly stressed. For a radar or communication tower, loss of orientation may be critical. If the tower is guyed, the guys and/or the guy anchors may be overstressed. Some may become slack. Differential footing settlements may occur during thaw-weakening in spring. If the tower is on a slope, progressive downslope movement may occur with successive cycles of freeze-thaw.

c. Granular material may be used as illustrated by figures 4-89 and 4-91 to control or even eliminate detrimental vertical movement. The simplest approach, as shown in figures 4-89a,b, is to support the tower on a granular mat placed on the surface. Because of the intensity of the winter cold, it is usually impractical in arctic and subarctic regions to attempt to make the mat thick enough to completely prevent frost penetration or heave in the underlying frost-susceptible material, particularly when the mat is naturally well-drained, as in figures 4-89a,b, though this may be possible in some seasonal frost areas. However, as described in paragraph 2-5, the magnitude of frost heave may be substantially reduced by a relatively modest surcharge, consisting of the weight of the gravel plus the load from the structure. For some situations, the thickness of gravel may therefore merely need to be made sufficient to reduce frost heave to an acceptable level, assuming the design is not sensitive to possible differential effects.

d. In the type of design shown in figure 4-89b, the footings are placed at the natural ground surface instead of on the mat. If the mat densities and thicknesses are the same, the potential for heave reduction by surcharge will be the same in figure 4-89a and b. However, safety against overturning of the self-supporting tower will be greater in figure 4-89b because of the load of the mat on the footings. There is greater possibility in figure 4-89b that the soil under the footing may be overstressed during spring thaw weakening. To reduce this possibility, as well as to provide a working surface and to make sure that footings will not get "hung up" and fail to settle completely back to original position on thaw, it is desirable to specify a shallow granular pad immediately below the footing as shown in figure 4-89b. More steel is required in the figure 4-89b scheme than in the figure 4-89a design, placement of gravel around the structural members requires special care, and protection of the buried steel against corrosion is more complicated; this scheme is thus more expensive.

e. If in the figure 4-89b design the footings were to

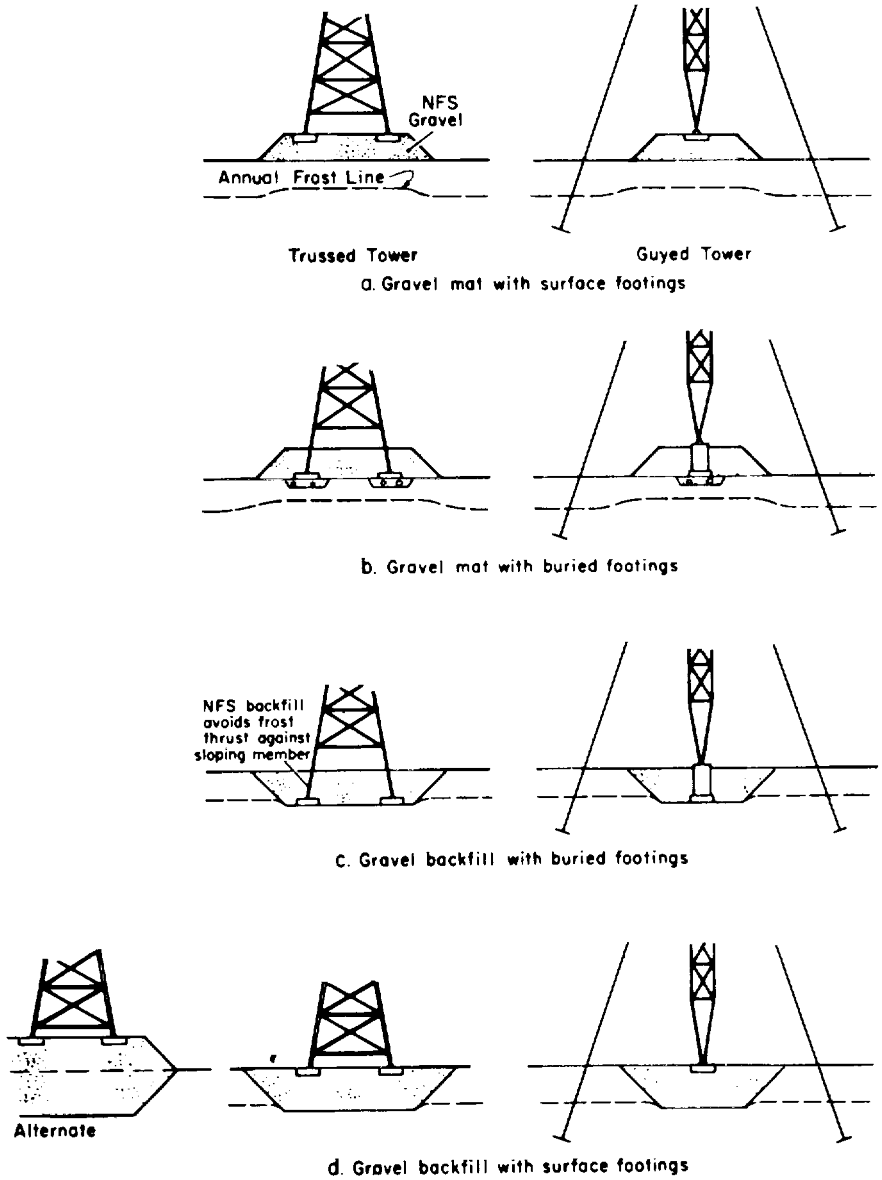

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Figure 4-89. Granular pad tower foundation.

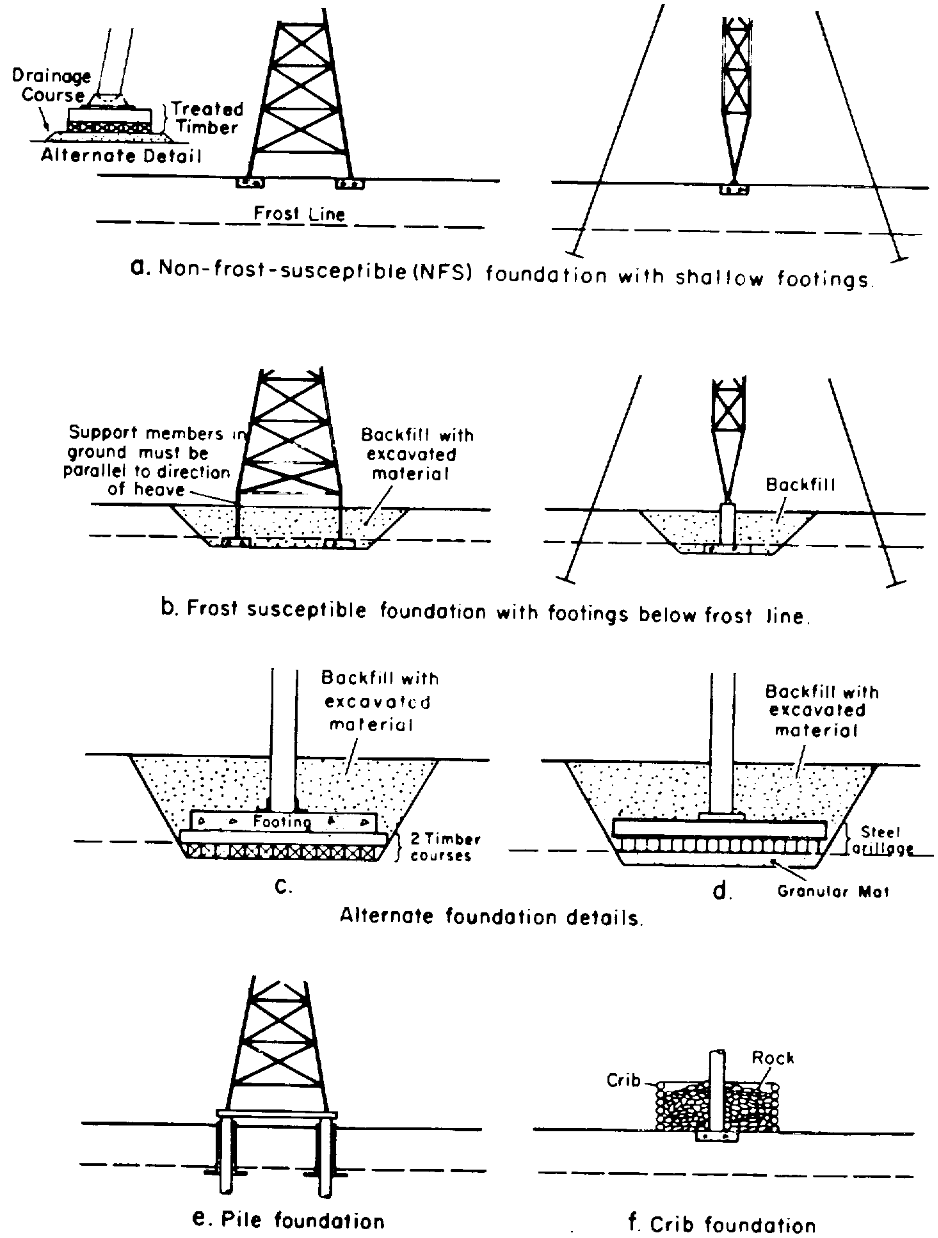

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Figure 4-90. Foundation designs employing minimum or no NFS granular borrowed material (both are frost-susceptible foundation).

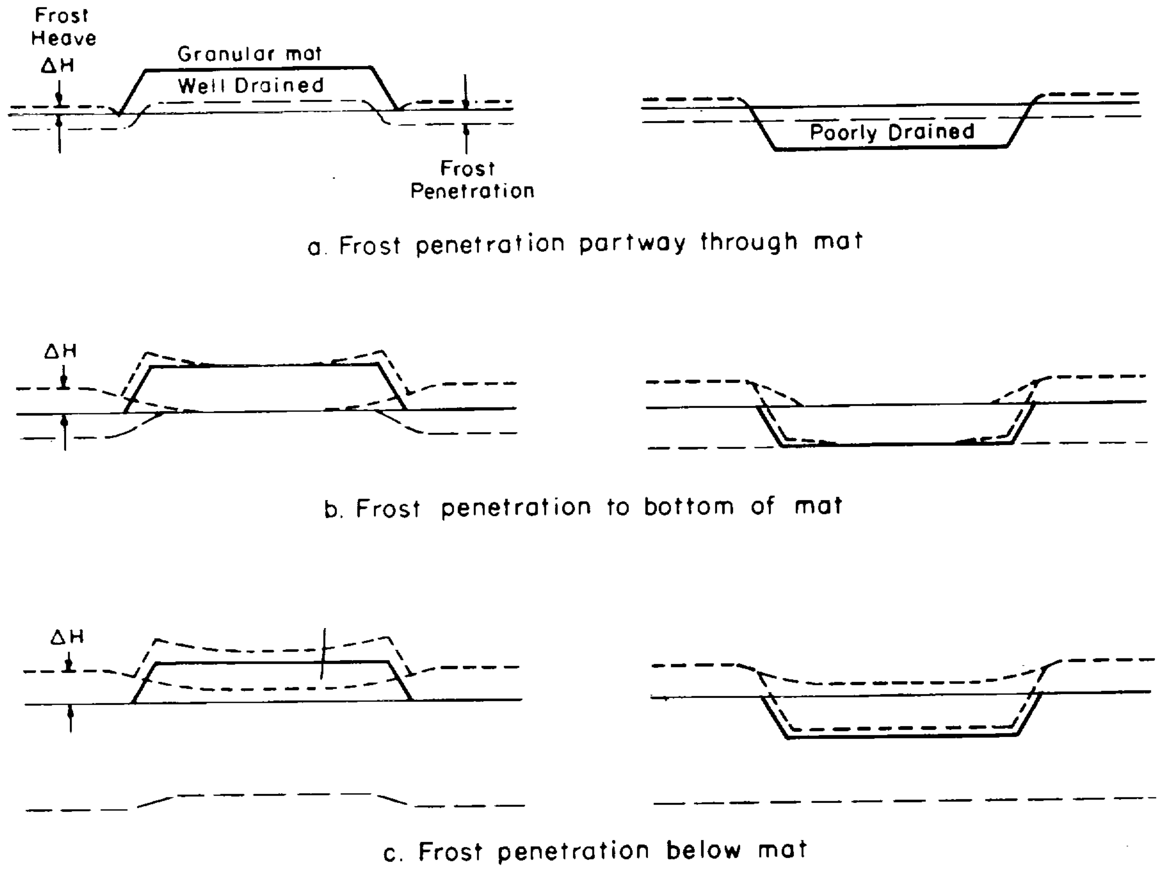

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Figure 4-91. Effects of granular mats on frost penetration and heave.

be placed within the frost heaving material, the possibility would exist that the footing would be moved progressively upward with successive annual cycles of freeze and thaw, in the same way that boulders work upward in the seasonal frost zone. Therefore, where foundation soils are frost-susceptible, footings must be placed either on top of the frost-susceptible material or granular mat or below the zone of seasonal frost, never in between. If in the case shown in figure 4-89b, the footings were to be placed below the seasonal frost zone, care would have to be taken that the direction of frost heave and the axes of the structural members within the annual frost zone had the same orientation so that the heaving soil could "slide" on the structural members. In hilly country this requirement might be impossible to achieve. Of course, the structural members would also have to be free of projections or obstructions and would have to be designed, together with the footings, to resist the heave forces generated in them.

f. The same concepts as involved in figure 4-89b are also represented in the approach shown in figure 4-89c. On the plus side, the latter design offers the additional advantages that moisture content of the granular fill will tend to be higher and the total seasonal frost penetration less than in the figure 4-89b case, because of the poorer NFS drainage situation, the footings rest on material not subject to freeze and thaw which thus will retain its bearing capacity and snow cover will develop relatively normally because through the year the backfill is flush with the surrounding ground. However, the scheme has the inherent disadvantages that the additional cost of excavation and removal of material in the annual frost zone is bound to make this scheme more expensive than that in figure 4-89b (which in turn is more costly than that in 'fig 4-89a), and unless the mat is of substantial diameter there is a real possibility that the frozen moist granular mat, forming a relatively continuous slab with the adjacent frozen soils, may be lifted by heave of these surrounding soils. If such lifting is possible, careful analysis of possible frost heaving forces on the bottom structural members of the tower or of possible lifting of the tower itself will be required.

g. The center and right hand diagrams in figure 4-89d show the same mat and footing arrangements as in figure 4-89a except that the mat is placed below the surface, to gain the advantages of higher moisture retention potential and corresponding reduced frost penetration. The disadvantages are the same as explained above for figure 4-89c except that possible heave forces on the non-perpendicular bottom structural members are avoided. The self-supporting design also lacks the added safety against overturning provided by load of the mat on the footings. The left hand diagram of figure 4-89d illustrates how the design might be further combined with the mat in figure 4-89a to further increase the surcharge and better control frost penetration.

h. The development of frost penetration and frost heave in above-surface and below-surface granular mats is illustrated diagrammatically in figure 4-91 for three relative depths of frost penetration, disregarding possible effects from non-uniform snow cover. In figure 4-91a, freezing has only partially penetrated the mat and there is no frost heave in the interior of the mat. In figure 4-91b, frost penetration has reached the bottom of the mat but frost heave of the mat is confined to the shoulders. In figure 4-91c frost has penetrated below the mat; the entire mat has lifted and the surface is dish shaped. At the same time heave of the natural ground tends to be restrained near the mat; heave at the mat is less than it would have been without the mat. Because the transition zone conditions at the boundary of the mat develop gradually over the winter, upward bending of the frozen granular material may occur by creep.

i. It will be apparent from these diagrams that tower footings should be located a prudent distance away from the mat boundaries in order to minimize frost heave problems. No tower footing edge should be closer than 5 feet to the top edge of the granular embankment under average conditions.

j. Although some guidance in selection of mat thickness is given in paragraphs 2-4, 4-2b, and 4-2e, it is difficult to estimate the actual maximum frost heave accurately, even at the center of the mat, in the present state-of-the-art. For example, the projection of a mat above the surface, as shown in figure 4-89a, will affect the thickness and uniformity of snow cover developed locally, but the exact effect of the variable snow accumulation on frost penetration under the interior of the mat is difficult to predict. It is even more difficult to predict effects in the perimeter transition zone. For both above-surface and below-surface mats there is at present no rational technique for analytically determining the pattern of vertical deformation in this zone or of the diameter of mat required to isolate the footings from the upward thrust of the surrounding frost heaving materials.

k. Therefore, for designs which require frost heave to be predicted with a high degree of confidence, prototype/scale test installations should be constructed under representative field conditions; heave and frost penetration should be measured on these in at least one winter, correlated with soil moisture and freezing conditions, and projected to the worst anticipated winter conditions during the life of the structure.

l. Figure 4-90a shows, schematically, tower foundation designs for non-frost-susceptible foundation materials using surface footings. Since these are little affected by freeze and thaw except for thermal contraction and expansion of the ground surface, foundation designs may be essentially the same as in non-frost areas.

m. In figure 4-90b, footings in a frost-susceptible foundation are shown placed at a level below the zone of seasonal freeze and thaw. In seasonal frost areas, these footings may rest directly on natural soil. In permafrost areas a granular working surface of nominal thickness should be employed directly under the footing. By backfilling over the footings with the same material as that removed, the depth of seasonal frost penetration under the tower will experience minimum change from the natural conditions, the major remaining cause of difference then being the effects of destruction of the surface vegetation during construction. If the soils have high moisture holding capacities, the depth of frost penetration, and hence the needed depth of excavation, will be a minimum. As shown in figure 4-90b, however, the structural members passing through the frost zone must parallel the direction of frost heave; this requirement may make this type design impractical in hilly country. Sometimes it may be possible to modify the topography locally, in the area of the foundation, sufficiently so that all frost penetration will be vertical.

n. Figures 4-90c and d show details of two possible alternate footings. Figure 4-90c illustrates the use of two timber courses to provide a firm, semi-insulating working surface and footing base. In some areas, timber may be more readily available and more easily handled than gravel. Figure 4-90d shows a steel grillage resting on a shallow gravel working course. If the backfill is frost-susceptible, frost heave forces acting on the vertical member of the foundation in figures 4-90c and d must be analyzed and the footings designed so that they offer adequate resistance against being pulled upward in winter.

o. Figure 4-90e illustrates a pile foundation with flanged sleeves to isolate the piles from frost heave forces. The sleeves may be omitted if adequate pile embedment in permafrost or other provisions against uplife are provided. Sleeves and other techniques for providing heave force isolation are discussed in greater detail in paragraph 4-3/.

p. The timber crib foundation shown in figure 4-90f has been used successfully for pole lines in difficult marginal permafrost terrain. Pole lines are further discussed in TM 5-852-5[13].

4-12. Bridge foundations.

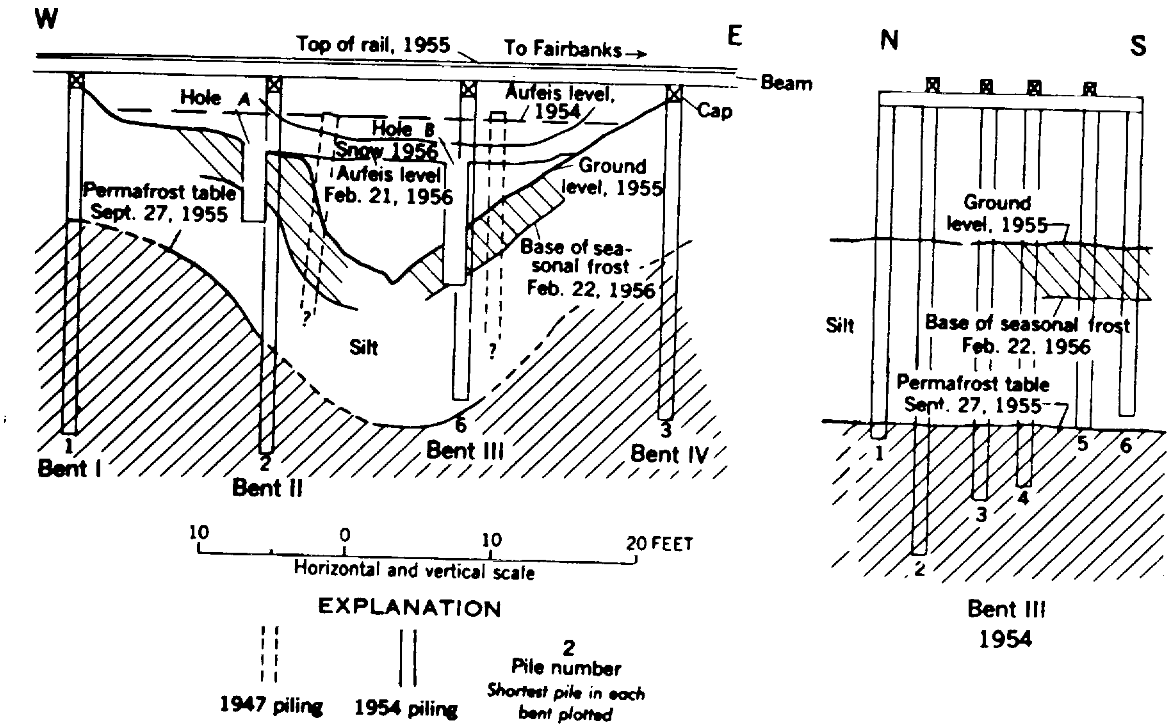
a. Foundations of bridges which do not cross water bodies should be designed using the previously described criteria for walls and retaining structures (para 410), footings and piers (para 4-7), and piling (para 4-8), as applicable.

b. Bridges over water bodies in permafrost areas tend to involve difficult special problems because the permafrost conditions are substantially altered near and under the water. As shown in figure 4-92, the permafrost table tends to be depressed under a water body; under a major water body, permafrost may be absent except very close to the shore. Temperatures of permafrost near and under the water also tend to be warmer; especially in areas of marginal permafrost, permafrost temperatures at the ground levels in which foundations are supported may be very close to or at the melting point. Water moving in thaw zones beneath and adjacent to the water body may cause an extremely complex and uncertain thermal regime pattern. Little or no capacity for natural freezeback of piles may be available and the tangential adfreeze strength that can be safely relied on may be very low. Footing-type foundations encounter substantial risk of settlement from slight changes in the subsurface thermal regime.

c. For all these reasons highway and railroad bridge foundations in permafrost areas have been a continuing source of difficulties. In marginal permafrost areas, designs of stable piers and abutments are among the most challenging engineering problems which may be encountered in permafrost areas. In order to support these facilities on relatively stable frozen materials, pile foundations are commonly employed[48,50]. However, because of uncertain or incomplete freezeback, the frequency of serious frost heaving of pile bridge foundations has been very high. On the Alaska Railroad for example, wood piling of many of the bridges is heaved every year. Pewe[180] has reported that this heave reaches as much as 14 in./yr and the elevation of the track is seriously disturbed, making necessary reduction of speed to avoid uncoupling of cars or shifting of cargo. Tops of piles are trimmed off each summer, resulting in progressive reduction of embedded length. Maintenance requirements are substantial and periodic replacement of piling and even changes of alignment are required. At some bridge sites, the stubs of several "generations" of piling which have been successively abandoned may be seen, d. Especially careful and detailed subsurface exploration should be carried out at proposed bridge locations to assure the most favorable bridge alignment and positioning of the foundations and to provide the information required for thorough and painstaking foundation design of these vital facilities. Positions of the permafrost table and any residual thaw zones should be carefully determined. The amount of any excess ice in the ground should be carefully ascertained using refrigerated drilling techniques, and the subsurface temperature conditions, should be determined.

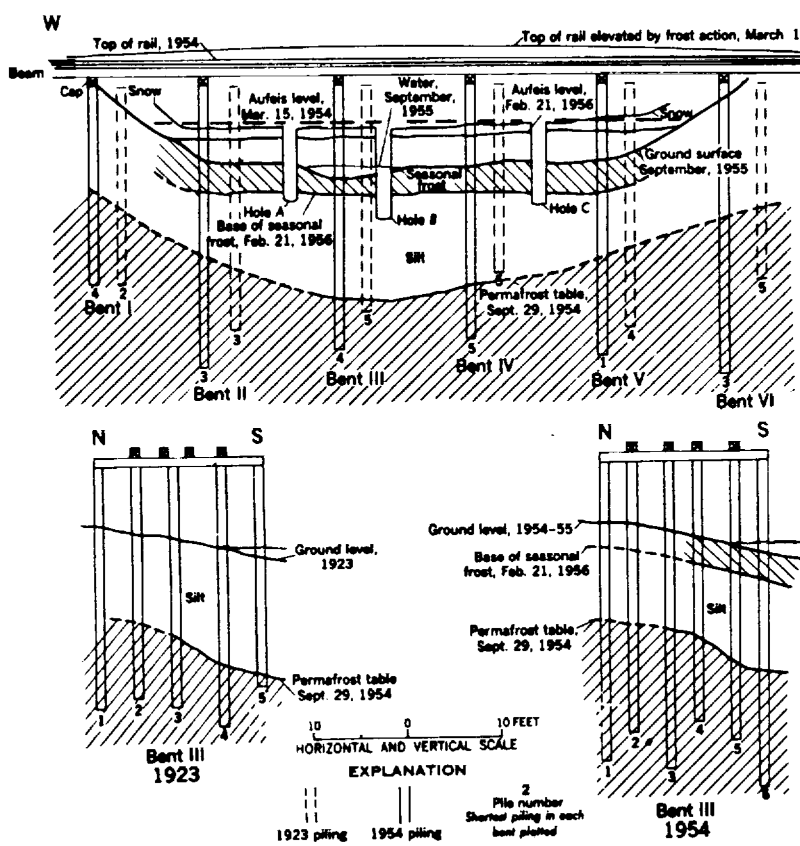
e. The presence of frozen ground should not be assumed to preclude scour; fine-grained permafrost soils are often very highly susceptible to erosion upon thaw and the effects of floods may be substantial, particularly directly adjacent to piers placed in the streams. Also, the gouging action of floating ice often has a powerful eroding effect on stream banks. Since stream flow and thermal regime patterns may change substantially and unpredictably over the life of a bridge, and hydrologic data are often grossly inadequate, bridge foundations should be placed at depths which are conservatively safe with respect to usual criteria for safety against undercutting. Stationary ice sheets"', ice jams, and ice sheets and ice cakes moving with various velocities of flowing water or blown by wind and with various angles of attack, can exert very substantial pressures on foundations placed in the water. Foundations and piers should be positioned and shaped so as to minimize the effects that these forces can exert on exposed members of the foundation and designed with sufficient armor and strength to resist the forces which may then still occur. Michel[78,79] and Dunham[140] present useful discussions of this problem. Davis has reported on rock fill and sheet pile construction exposed to sea ice in Thule Harbor[54]. Techniques for design of structures against ice forces are in early stages of development.

f. Icing or the progressive accumulation of ice in winter by freezing of seepage or stream flow on the surface is unlikely to contribute any structural loadings to the foundation if the ice rests directly on the ground, although it may significantly reduce the hydraulic capacity of the bridge opening. However, if the ice build-up occurs on floating ice, a downward thrust on the foundation may be exerted in winter because of



(Courtesy of U.S. Geological Survey)

Figure 4-92a. Geologic sections at two Alaska railroad bridges in Goldstream Valley near Fairbanks, Alaska. (From Pewe[180].) (Milepost 456.7.)



(Courtesy of U.S. Geological Survey)

Figure 4-92b. Geological sections at two Alaska railroad bridges in Goldstream Valley near Fairbanks, Alaska. (From Pewe[180].) (Milepost 458.4.)

adherence of the ice to the pier, either from the increasing accumulation of ice above the water level or by lowering of the water level with decreasing stream flow. On the other hand, raising of the water level in the spring can cause a large upward thrust on the pier from the buoyancy of ice adhering to the pier or bearing against connecting members. In tidal areas the adherence of sea ice sheets to piers or piles at low tide may not only cause spalling or other direct structural damage but may cause them to be jacked up by the buoyant force of the ice in the succeeding high tide. In coastal Connecticut, piles for a waterfront structure were pulled out of the ground in one winter by this action. When such uplift is possible, piers, caissons, or piles should be kept entirely smooth, without projection or cross-bracing, in the levels at which ice may act, and must have sufficient depth of embedment or other anchorage to resist such uplift forces as can develop.

g. If the foundation is supported on frozen ground, short duration forces such as wind gusts are likely to be of little consequence because design for long term stability will have automatically introduced large factors of safety relative to short term loadings. However, any loadings which can act relatively consistently over substantial periods must be taken into account in the design load assumptions. In all cases careful analysis of frost heaving forces is required (para 4-31) and a safety factor against heave must be provided (para 4-8h). As shown in figure 4-41, which represents a relatively stable bridge for subarctic conditions, bridge foundations tend to be continuously in motion due to seasonal effects. If progressive frost heave occurs, or if allowable bearing stresses are overestimated and settlement occurs, movements may be progressive and much greater in magnitude, as well as differential. Because of these ever-present possibilities, types of bridge span structures which are especially sensitive to foundation movements should not be employed in arctic and subarctic areas unless stable foundations can be provided with complete certainty.

4-13. Culverts.

a. Criteria for design of flexible and rigid pipe culverts, for required depths of placement of culverts and of required depths of cover below pavements are given in TM 5-820-2/AFM 88-5, Chapter 1[8] and TM 5-820-3/AFM 88-5, Chapter 3[9]. The problem of icing in culverts is discussed in TM 5-852-7/AFM 88-19, Chapter 7[17].

b. The problems of thermal stability of culvert foundations are somewhat similar to those of bridges in that the presence and flow of water causes a special thermal regime under the structure. If the culvert is a large one and carries water flow during most of the year, the frost penetration pattern may be substantially altered locally. If the culvert is constructed in a natural drainageway, a special, relatively stable thermal pattern may already exist before construction and a condition of substantial seepage flow through the soil under the culvert location may already have been established. For this reason a natural drainage site is nearly always preferable. If the culvert is cut into an area which has not previously carried such flow, a new thermal regime will begin to develop; if the culvert is cut into permafrost containing ground ice, thaw will occur in summer into the soil surrounding the culvert structure from the effects of both the heat in the water flowing at the bottom of the culvert and the exposure to above-freezing air temperatures. Since the surface of ponded water exposed to the sun in arctic and subarctic areas may reach temperatures as high as about 70°F in the summer, heat input from the water may be substantial. If permafrost containing ground ice underlies the culvert, catastrophic settlement can be produced in a single summer. Not only may the culvert structure be damaged by loss of support, but the water may begin to pass under the structure instead of through it, leading to even more rapid collapse. If the flow is derived primarily from snow melt or emergence of seepage from thawing ground, it may be close to 32 F. If the water before reaching the culvert is experiencing net heat loss by radiation to the sky, it may even contain frazil ice particles.

c. In truly arctic areas it is possible to construct a stable culvert cut into permafrost by placing sufficient non-frost-susceptible backfill under and around the culvert structure so that thaw will be confined to this material in summer, with thawed material refreezing in the following winter. Required depth of gravel may be computed by analysis methods outlined in TM 852-6/AFM 88-19, Chapter 6". It will be necessary to determine not only the shaded air temperature but also the temperature of the water flowing in summer in the culvert and to compute thaw penetrations separately for these conditions using techniques for two-dimensional radial heat flow analysis. The only accurate way of determining the temperature of water which is flowing in a culvert is by actual measurement during a summer season. When in doubt, the designer should make certain that any error is on the safe side. Insulation may sometimes be economically substituted for part of the non-frost-susceptible material. In marginal permafrost areas, however, it may not be possible to achieve essentially complete freeze-back in winter and thaw will then be progressive. Insulation can slow but not prevent this. In this case, if unacceptable settlement would otherwise result, the designer should consider use of only established drainageways where reasonable thermal stability has been naturally achieved, use of pile-supported bridges instead of culverts, or complete excavation of the thaw-susceptible materials and replace ment with non-frost-susceptible material. For nonpermanent facilities it may sometimes be necessary to accept heavy maintenance costs, however.

d. If foundation soils are susceptible to settlement on thaw, thermal analysis should be made of the proposed culvert, adapting the methods of TM 5-852-6/AFM 88-19, Chapter 6 [14]. The final design should provide a thermally stable condition.

e. Headwalls of culverts may be heaved, undercut, tilted and fractured by frost action unless properly designed and constructed. They should be designed using the same structural approaches as outlined in paragraph 4-10, and with conventional provisions against piping through unfrozen material under the culvert.

4-14. Anchorages.

a. Economical construction of anchors in frozen ground is a difficult and challenging problem because of the marked tendency for anchors to yield and creep when anchoring in a frozen soils' and also because of frost heave forces within the annual frost zone. While anchors in frozen soil may be capable of sustaining relatively high, short-duration loads without difficulty, they can exhibit unacceptable yield and creep under much lower long-term loadings. The latter is most pronounced when frozen ground temperatures are only slightly below the freezing point. Possible long term or transient changes in thermal regime must be carefully evaluated.

b. Laboratory experiments have shown that plate type anchors entirely in frozen soil (no thawed layer at the surface) fail in two distinct modes depending on the depth of burial. When the ratio of depth below surface divided by diameter of plate is greater than six, failure of the anchor occurs by punching of the plate on a cylindrical surface through the overlying material in a manner similar to that by which an overstressed footing may punch downward into a foundation. This is illustrated in b through f of figure 4-93. At stresses less than those which will produce rupture under relatively rapid loading, creep deformation will occur under long term loading in the same manner as described for footings in paragraph 4-7. When the plate is buried less than six times the diameter of the plate below the surface, failure may be expected to take place by punching out of a truncated cone of material starting at the plate and widening out at a 30° angle from the vertical, as shown in figure 4-93g. As this cone approaches the ground surface, the angle may change abruptly to perhaps 20° with the ground surface. However, it is conservative to assume in the analysis that the 30° angle continues to the surface. If the anchor acts at an angle with the ground surface, the failure surface may be expected to be altered as illustrated in figure 4-94.

c. Therefore, plate type anchors may be analyzed in the same manner as footings when the depth of burial in frozen material exceeds six times the plate diameter; soil stresses on the anchor rod should be included in the analysis. Anchors closer to the surface than six times the diameter of the plate may be analyzed in terms of stresses on an assumed 30° truncated cone. In both cases, the presence of a thawed layer at the surface, of distinctly different characteristics from the underlying frozen material, will usually require that it be handled as a separate element within the analysis.

d. Conventional plate, screw-in type or various patented earth anchors can be installed in inclined or vertical augered holes in permafrost with slurry or backfill of soil-water mixture, compacted moist soil, or crushed rock. The capacity of such anchors is greatly increased by freezing and keeping the backfill frozen. If the anchor relies significantly upon the strength and resistance of thawed soil after installation, all efforts should be directed to selection of the most suitable backfill and the attainment of good compactive effort.

e. In recent years, special helical anchors have been developed for installation in permafrost. These anchors often have multiple helixes increasing in diameter from the bottom and are designed to resist the large torques required for installation in frozen soil by truck-or crane-mounted power equipment. Conventional helical anchors used for unfrozen soils may fail during installation by shearing the rod from the helix.

f. The design capacities for the various sizes and shapes of commercial earth anchors, as published in various tables in handbooks or manufacturers' literature for a range of unfrozen soils, should be reduced by 75 percent for anchors in thawed soil above permafrost. Unless protected in the annual frost zone by anti-heave devices or treated backfill, all anchors embedded in permafrost should be designed so that the anchor rod is capable of resisting 60 psi of frost thrust within that part of the rod which will be in the annual frost layer, and a total frost uplift force would be computed by assuming the average of 40 psi acting over the depth of the annual frost zone; the latter should be added to the design tensile load imposed on the anchor. For anti-heave protection, see paragraph 4-31. Provisions should be made for adjusting the tension of guy lines in both summer and winter, since the pole, tower or structure being guyed and anchored may experience heave or settlement quite different from that of the anchor(s).

g. Conventional metal expanding anchors should not be used in frozen soil or in rock containing ice as the extremely high local stresses developed with such anchors cause rapid plastic deformation and creep in the ice component. Only anchors which develop very low level

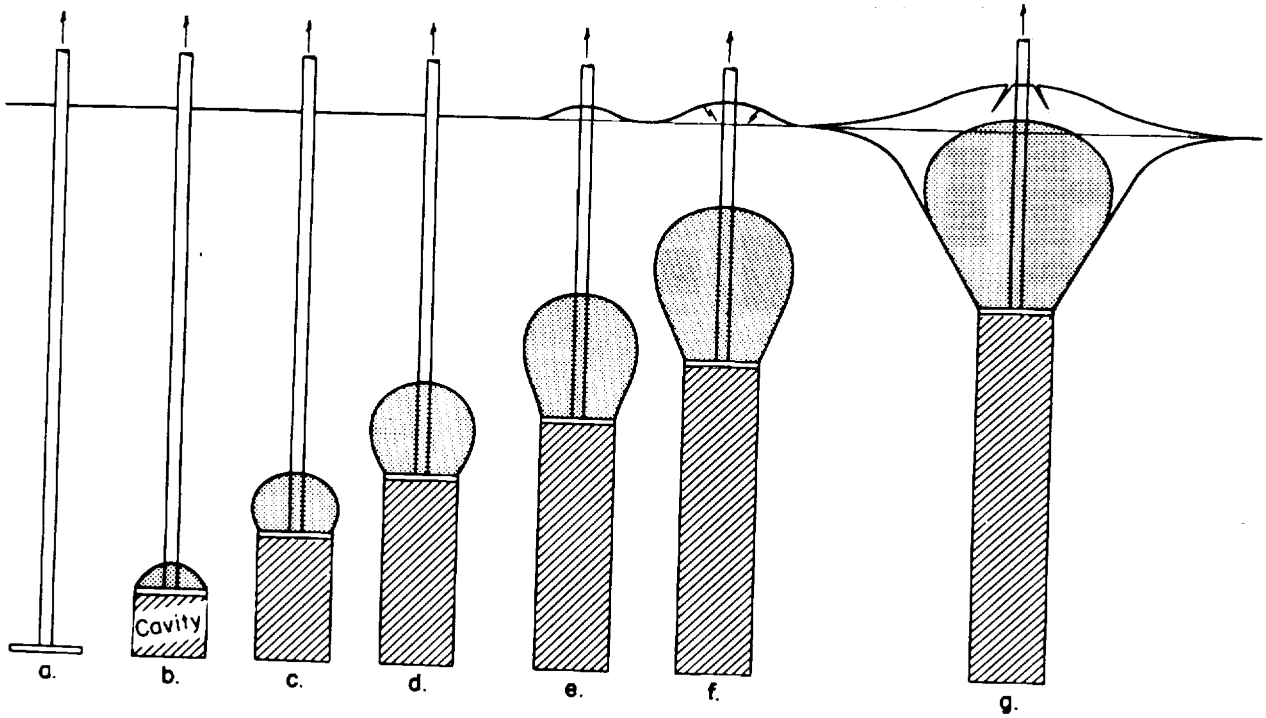


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Figure 4-93. Mechanics of anchor failure in frozen soils.

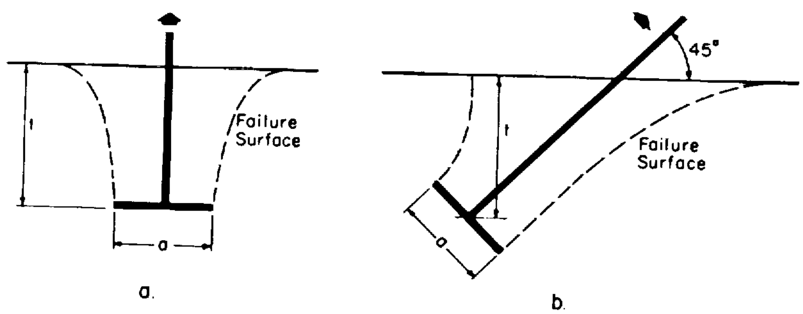

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Figure 4-94. Failure planes for batter and vertical anchor installation.

stresses over a relatively large area should be used. These are essentially the same principles as used in design of pile foundations in frozen ground (para 4-8).

h. Grouted anchors may be set in ice-free rock in conventional drill holes. The drill holes will require preheating before grouting if the rock is frozen. Grouted anchors may be installed in ice-free rock without preheating if the rock is warmer than 30°F, if high-early or other fast setting cements are used, provided the temperature of the ground is greater than 60°F at the time of placement and the annular thickness of the grout around the anchor rod is at least 2 1/2 inches (i.e., diameter of hole 6 inches or greater for 1-inch rod). Enlarged bells may be augered and integrally poured with the normal grouted rod anchors to provide additional anchor capacity. Because of the low ground temperatures, lead was used to grout anchorages into bedrock during construction of a major antenna at Thule, Greenland, in the 1950's to avoid the uncertainties of using portland cement mortar under these conditions. However, such practice is not recommended today in light of present techniques and capabilities for analyzing such problems.

i. Mass-gravity anchors have the advantage in cold regions that they are positive, can always be counted on, and are free of the risk of creep. However, if placed on top of frost-susceptible soils, the risk of frost heave and consequent variable anchor tension must be considered. Mass-gravity anchors are particularly suitable where clean, granular or even bouldery soils exist which can be easily excavated and handled for complete or partial burial of cast-in-place or precase anchor units. Deadmen can also be advantageously used to bear against frozen soil, but excavation costs are usually quite high.

j. In permafrost areas it is usually preferable to install anchors in the winter in order to cause as little permanent thermal disturbance of the permafrost as possible while at the same time assuring rapid development of the design anchor capacity.

k. For permanent anchors in frozen ground, design should be predicated on whichever is controlling: ultimate strength or holding creep within acceptable limits. Factors of safety should not be less than those specified in paragraphs 4-4 or 4-8h depending on the type of stressing. Failure of an anchor by pull-out is more likely to be catastrophic than the failure of a footing in settlement would be. Therefore, the factor of safety against actual pull-out should also be at least equivalent to the factor of safety in the supported structure based on ultimate strength.

4-15. Foundations for non-heated facilities.

Foundations of non-heated facilities may involve special design problems, considerations or requirements.

a. *Non-heated buildings.*

(1) Temperatures at floor level in an unheated building will depend on such factors as roof and wall insulation, degree of ventilation, roof and wall exterior reflectivity, and seasonal percent sunshine and are best determined experimentally in comparable buildings in the same area. Temperatures at floor level in an unheated building fully open to the outside air may usually be assumed to average the same as standard shaded meteorological station air temperatures. For this situation, slab-on-grade construction without insulation can be employed if a mat of non-frost-susceptible material is used, as illustrated in figure 4-95, sufficiently thick to contain seasonal freeze and thaw. With modification of the thickness of the non-frost susceptible mat as required by local climate, and possibly radiant heat input through windows, this design can be used under any building in any seasonal frost area or under any fully ventilated, unheated building in any frost area. Closed unheated buildings with no more than nominal ventilation tend to have warmer average annual temperatures, particularly from absorption of solar heat in summer. This is qualitatively illustrated in figure 4-40. Degradation of permafrost under this closed, insulated building (with ineffective foundation ventilation system) only slowly decreased after discontinuance of heating. If the average annual temperature in the building is warm enough to cause degradation of permafrost, the design in figure 4-95 will no longer be suitable if the foundation soils will settle significantly on thaw. Temperatures in an unheated earth-covered igloo or below-ground structure may usually be assumed to average the same as the ground temperatures at the average depth of the facility. The possibility that lighting or other electrical facilities and body heat may introduce significant amounts of heat into closed facilities should be considered.

(2) Where non-frost-susceptible material is scarce or expensive or where very deep frost penetration would require an uneconomical thickness of mat, the following alternatives to the designs in figure 4-95 may be considered.

(a) Use of under-slab insulation to reduce the thickness of non-frost-susceptible fill required.

(b) Use of a structural floor supported by footings or piles sufficiently above the ground so that it will be isolated from frost heave. This system can also be used to provide foundation ventilation to insure the coldest possible temperature conditions at the ground surface.

(c) Use of a gravel floor of nominal thickness directly on the natural soil, accepting resultant frost heave.

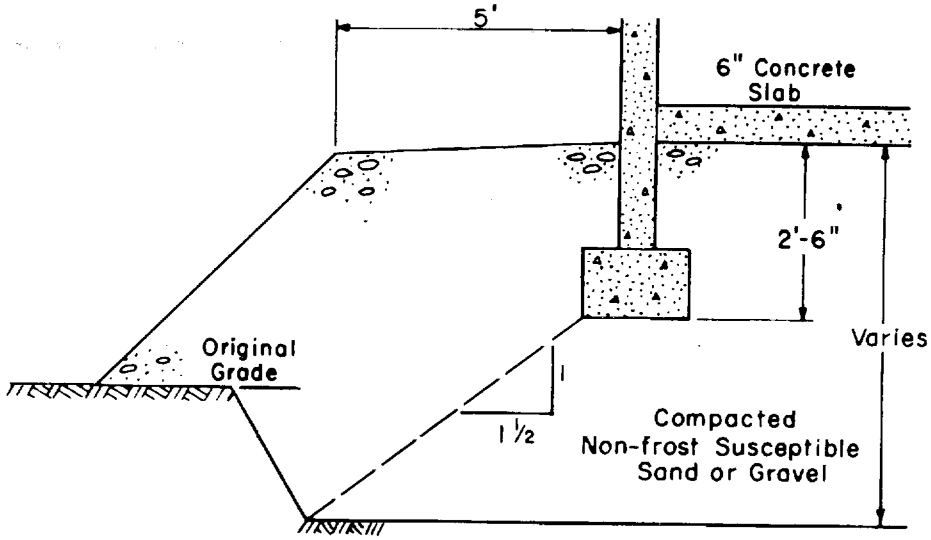

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Figure 4-95. Typical foundation design for unheated buildings overfrost-susceptible soil in deep seasonal frost or permafrost areas.

(d) Use of a mesh-reinforced concrete floor slab with sufficient non-frost-susceptible material to reduce total heave to about 1 inch (assume l/2 of this differential), accepting some slab movement and fine cracks.

b. Exterior footings and piles.

(1) Footings and piles placed on the exterior of heated buildings for support of porches, roof extensions and unheated connecting corridors, and which receive none of the heating benefits experienced by the main foundation, are subject to full frost-heave effects. In fact, because snow cover may be absent, frost penetration may be more than it is farther from the building where snow is allowed to accumulate. The importance of adequate provisions against heave of such footings is frequently overlooked, perhaps in part because the construction measures required seem out of proportion to the importance or construction cost of the facilities involved. However, the cost of repair measures for structural damage, blocked roof drainage, broken glass and distorted structures may substantially exceed the cost of adequate initial protection against heave.

(2) Foundations of this type should be constructed in accordance with the principles outlined in paragraphs 4-3, 4-7 and 4-8. If only small pipe columns are required, they may be installed inside protective casings extending through the annual frost zone with the space between casing and column filled with an oil-wax mixture which will permit free relative vertical movement of the casing and column but prevent entry of water and dirt; the casing should have an external flange at its bottom end to minimize its tendency to gradually work out of the ground. The flange should be strong enough to resist an adfreeze uplift force on the outside of the casing of magnitude as indicated in paragraph 4-8. If the casing and oil wax are not used, the column should be fastened to a plate or footing of size sufficient to develop the passive resistance required to counter the frost heaving forces.

c. Exterior aprons.

(1) When an exterior unheated loading platform, apron, or transition pavement over frost-susceptible soil is connected to a structure which is heated or is otherwise protected against frost heave, difficulties frequently arise. As indicated in figure 4-96a, heave may cause an unacceptable, abrupt displacement of the apron at the junction with the building and may block outward opening doors. It may also cause structural damage, interfere with drainage and thus cause icing. A pad of non-frost-susceptible soil placed under the apron to the full depth of frost penetration and tapered in thickness as shown in figure 4-96b or c will eliminate this difficulty if the material can be kept well drained. If the nonsusceptible material cannot be drained and becomes saturated, however, some uplift can still occur as a result of expansion of water which is trapped in the voids, on freezing. This heave may still be sufficient to block outward-opening doors if the fill is deep or is borderline in its non-frost-susceptibility and clearances are insufficient. In this case, it may be necessary to construct all or part of the apron in the form of a structural slab supported on one side of the foundation wall of the building, and, on the other, on footing, beam, or pile support, as shown in figure 4-96d, with sufficient space under the slab so that the heaving soil will not come in

contact with it. If a beam supported near the surface is used and the outer edge of the slab is heaved then hinge action will occur at the inner edge, but there will be no step displacement. In some cases, a layer of insulation under the slab may assist in providing the most economical solution. Another alternative way of allowing for minor heave is to provide a downward step as small as 2 to 4 inches with the apron left free to heave.

(2) In extreme climates it is considered preferable to hand doors of heated dwellings to open inward because of the possibility of blocking of doors by heave of exterior aprons, stairs, or platforms, or by heavy snow or ice, even though this is contrary to conventional fire safety regulations.

4-16. Utilidor and pipeline foundations.

Methods of supporting utilidors and pipelines above and below ground are discussed in TM 5-852-5/AFM 88-19, Chapter 5[13]. Techniques for design of the pile or other types of foundation construction and support described therein should be in accordance with the provisions of this manual.

4-17. Connection of utilities to buildings.

a. The manner and depth at which utilities (water, sewer, electric, communications, etc.) approach and enter building below ground may be a factor in foundation design. It is important that provisions be made so that utility lines will not be sheared by heave or settlement where they pass through foundation walls and that lines carrying water will not freeze. In purely seasonal frost areas, water lines 6 inches or less in diameter should be laid with invert 6 inches below the computed maximum frost penetration depth. Larger water pipes should be laid so that the top of the pipe is at the computed maximum frost penetration depth. In areas of very deep frost penetration it may be more economical, if the soils are non-frost-susceptible, to place the entire system of water pipes at nominal depth and provide continuous circulation and heat during the freezing season; for some situations insulation may also be used effectively if it is protected against moisture absorption. Because of wide variations in operating conditions, it is difficult to give a simple rule for determining the minimum depth of sewer pipes to prevent freezing. However, pipes located according to the above criteria for water pipes should nearly always be safe, as sewage leaving a building is normally appreciably warmer than the water supply entering the building. However, when water supply lines are allowed to waste continuously into sewer lines in extremely cold periods to prevent water line freeze-ups, the sewage flow may be abnormally cold. Factors affecting design of sewer lines with respect to freezing conditions are outlined in TM 5-852-5/AFM 88-19, Chapter 5'3 and TM

b. As illustrated in figure 4-97a, utility lines passing through the seasonal frost zone should be oriented parallel to the direction in which frost heave acts. Anchored frost isolation sleeves should be installed if materials are frost-susceptible. When utility lines enter a facility laterally below ground level as in figure 4-97b, they should be placed below the anticipated depth of seasonal frost penetration to avoid shear at the interface. When backfill is placed under utility lines it of course must be compacted in accordance with standard provisions to avoid settlement. However, precautions must be taken to prevent freezing and ice segregation in such fill during placement if the soil will later thaw. It is impractical to attempt to estimate the amount of such frost heaving and to pre-position utility lines in order to allow for consequent later settlement on thaw.

c. In permafrost areas utility lines most commonly run above the ground surface and connections into buildings are relatively easily effected, although specific provisions to permit relative movement are often needed. If placement of utility lines below ground is desired in permafrost areas the possibility of shearing of the lines by relative movement at the foundation wall must be considered. If the utility line is laid within permafrost, the possibility that permafrost degradation and foundation settlement may later occur must be carefully examined. If placement of the utility line in the annual frost zone overlying permafrost is considered, both thaw settlement and frost heave effects may have to be contended with, depending on the type of soil. If there is any possibility that such a below-ground utility line may be subject to shearing action at the foundation line, it must either be laid within a surrounding conduit of large enough diameter to isolate it from any possible shearing action or it must be brought above the ground outside the foundation and enter the building through a conventional above-ground connection.

4-18. Drainage around structures.

a. Considerable permafrost thaw damage can be caused to foundations by seemingly insignificant amounts of water entering or moving through unfrozen ground under and near structures. Where groundwater flow is a potential threat to thermal stability of foundations, a substantial analysis of groundwater flow may be required, including possibly the use of dye to trace directions and velocities. In fine-grained soils, seepage flow is slow and may amount to only a few inches or feet per year, but in coarse gravels flow in the annual thaw zone as high as about 2500 ft/hr has been measured[105] If such groundwater flow is in contact with a source of warm water such as a lake or pond, substantial disruption of thermal regimes and melting of permafrost may result. Water temperatures at the surfaces of shallow ponds and lakes in permafrost areas have

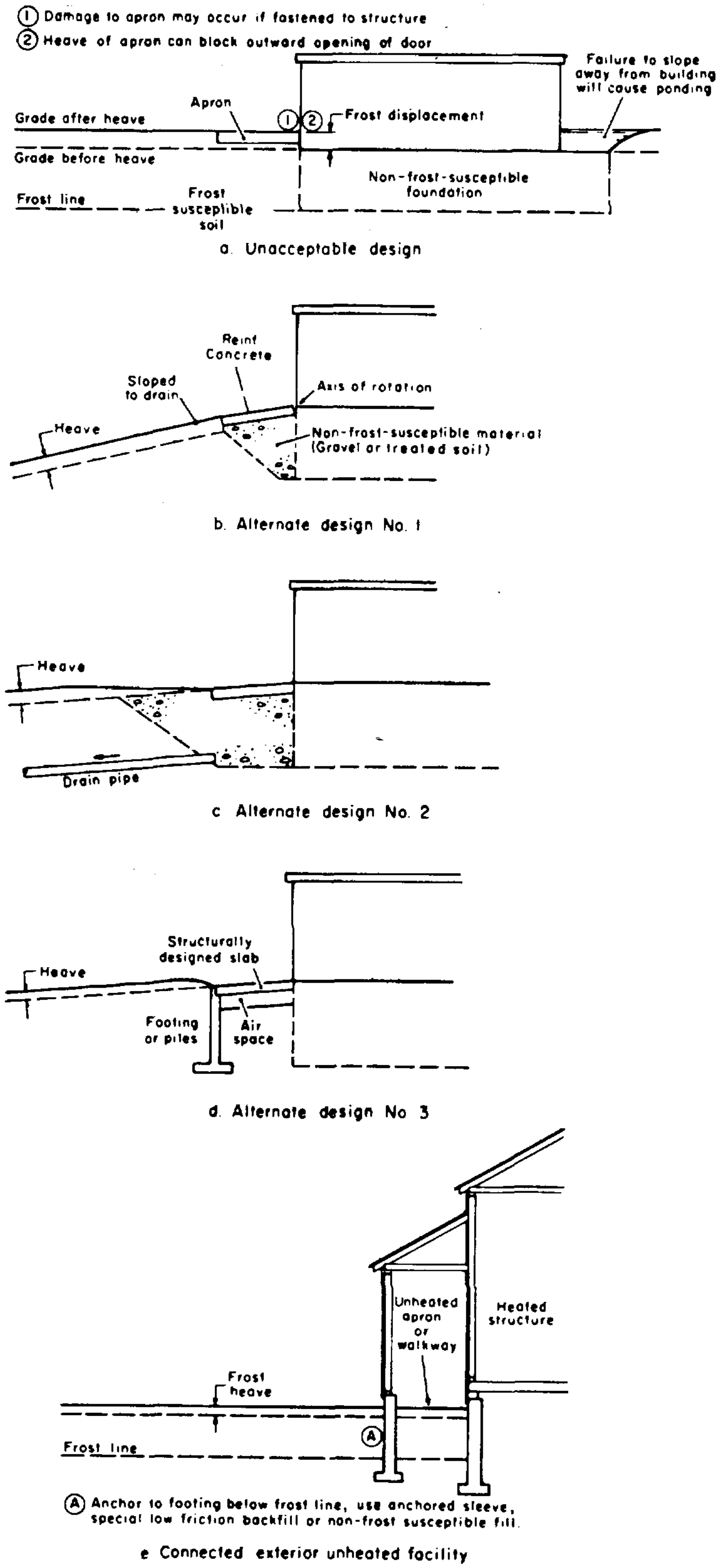

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Figure 4-96. Exterior apron design.

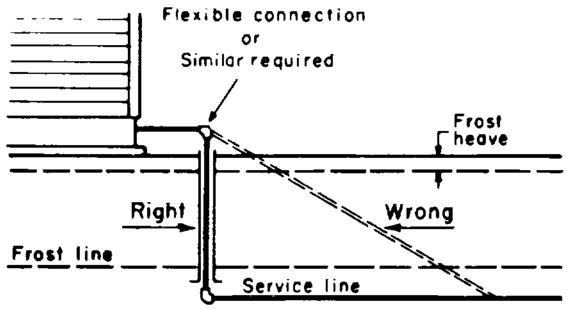

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Figure 4-97a. Utility connections to buildings. (Unheated facility on apron. Use flanged or otherwise anchored sleeve, special low friction backfill or non-frost-susceptible fill over sufficient area to prevent heave stressing of service line.)

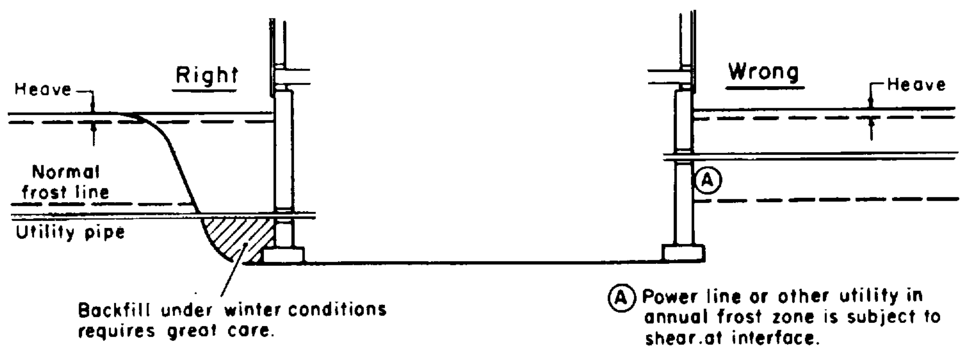

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Figure 4-97b. Utility connections to buildings. (Lateral utility connections.)

been measured as high as 70°F in the summer. Because of its high specific heat, even relatively cold water may have significant thawing capacity as demonstrated by the use of cold water to pre-thaw gravels and to remove frozen silt by sluicing in gold mining operations. In the warmer permafrost areas thaw zones readily develop under surface drainage channels; even temporary wastage of water on the surface during construction may produce thaw zones 10 or 20 feet deep which may remain unfrozen for decades thereafter. Wells drilled through permafrost should not be allowed to discharge indiscriminately on the ground surface in permafrost areas. Water absorbs solar radiation much more effectively than soil. Therefore, care must be taken in permafrost areas to slope surfaces at and near facilities so that surface water from snow melt or rainfall is drained away and ponding is positively prevented. Wastewater from buildings, particularly hot water such as waste steam condensate, must never be allowed to discharge on or into the ground near a permafrost foundation, even in small amounts. Good surface drainage is also important in seasonal frost areas to minimize frost action. In permafrost areas, natural subsurface seepage patterns in the annual thaw zone should be considered during site selection to avoid problem locations. However, it may also be possible to modify or control subsurface flow by judicious use of techniques for locally raising the permafrost table at critical locations, such as by placement of fill or use of shading or reflective surface color. One of the benefits sought from painting the runway white at Thule, Greenland, was the diversion of summer seepage flow in the annual thaw zone by raising of the permafrost table under the pavement to act as a dam[105].

b. Steam, water, and sewer lines must be kept completely tight. At an Alaskan facility minor leakage from an overhead steam line and resultant slow drip of condensate at the edge of the foundation contributed to thaw of permafrost to about 18 feet over a relatively short period.

c. Drainage ditches cut into ground underlain by permafrost containing ground ice or into permafrost itself should be avoided if at all possible because of the settlement and ground instability problems which will result from thawing. Thawing of ice wedges may lead surface drainage in unplanned directions. Undercutting and sloughing of drainage ditch slopes may cause silting and other problems. In soils or rocks capable of bridging, sing-holes and underground drainage channels may develop which may endanger even somewhat distant foundations. Under some conditions, it may be advisable to allow natural stabilization of the drainage effects to occur. This stabilization will occur most easily if ditches can be made shallow, penetrating only part of the annual thaw zone, rather than narrow and deep, TM 5-852-4/AFM 88-19, Chap. 4 even though shallow, wide ditches are more susceptible to icing. A 10 percent transverse slope should be used on the bottom of the ditch. When cutting into permafrost containing ground ice cannot be avoided and natural stabilization will not occur, cannot be relied on, or would involve unacceptable settlements and/or erosion, it may be possible to over-excavate the ditch and backfill to the desired cross-section with non-frost susceptible material of sufficient thickness to prevent summer thaw from reaching the underlying ice. On the other hand, it may be found that ditching at the site is simply impractical. An alternative then is to place the basic facilities on fill so that all need for cutting into permafrost is avoided.

d. Ditches in permafrost areas should be as short as possible. A reasonable slope is 0.003. Minimum width should be about 2 feet at the bottom with the actual width made sufficient to handle spring run-off. Side slopes of I on 2 are usually suitable. Where seepage into the ditch from side slopes will cause erosion and sloughing, where sloughing will occur as a result of thaw-weakening in spring and summer, or where control of permafrost degradation is required, blankets of free granular, non-frost-susceptible material may be employed on the slopes as discussed in paragraph 4-19.

e. Subsurface drainage systems, including trench drains, are not usually practical in areas of deep seasonal frost or permafrost unless they can be placed in ground which will be unfrozen at the time they need to function.

f. If cellars or basements are attempted under heated buildings in permafrost containing ground ice, the gradual melting of ice under and around the warm cellar will cause settlement not only of the building foundations but also of the surrounding ground. The results will be development of a dish-shaped depression surrounding and under the facility and an increasingly difficult water control problem in the cellar, which becomes in effect a sump for both surface run-off and permafrost melt water. An ordinary drain trenched from the cellar to a low point would soon freeze. Therefore, an endless problem of pumping and disposal of seepage water may be presented to which there is no good solution.

g. Drainage from flat roofs has often been piped down through the interior of buildings and into dry wells outside the foundation. Such systems have a history of problems in winter. The dry well and pipe drainage system outside the building commonly freezes up, and water backs up within the pipe inside the building. In summer, however, the contrary problem may exist of some local thawing of permafrost near the dry well from discharge of relatively warm water, heated on the roof. No good solutions to this roof drainage problem presently exist except discharge into the building sewer system in permafrost areas or, in seasonal frost areas, discharge into the ground at sufficient depth to be below the zone of freezing.

4-19. Stability of slopes during thaw.

a. Frost sloughing in areas of seasonal frost.

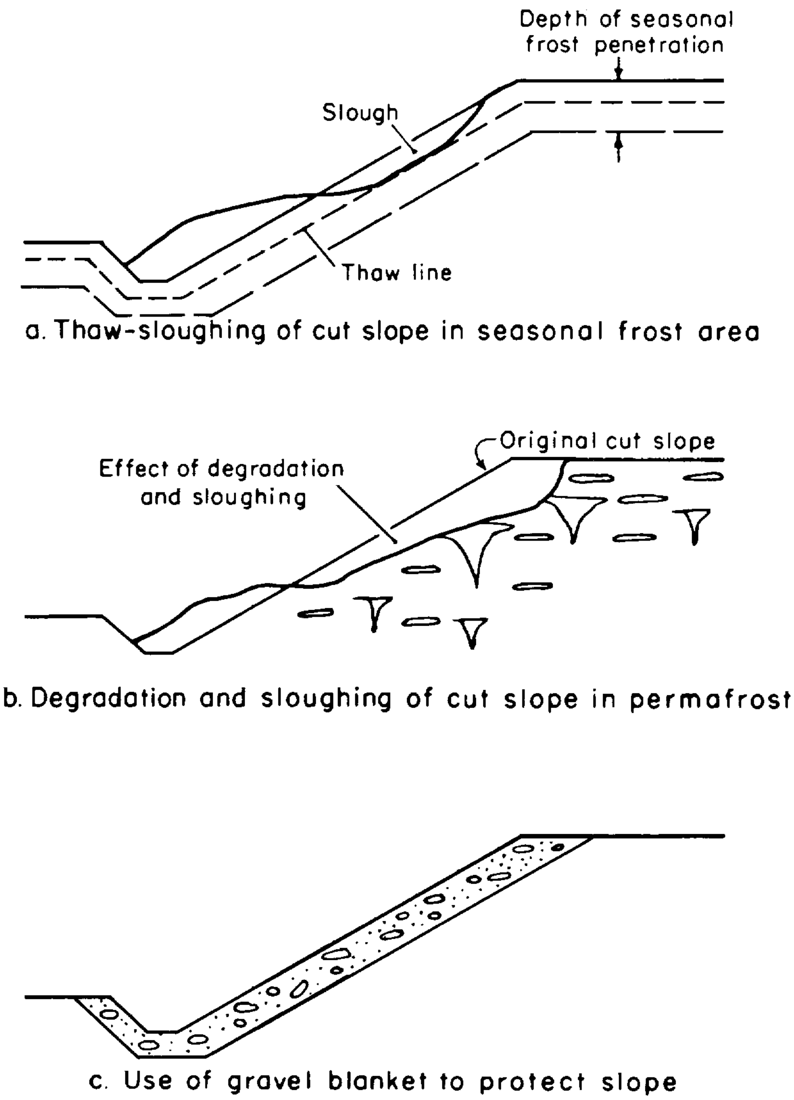
(1) In seasonal frost areas slopes composed of finegrained soils tend to experience "frost sloughing," as illustrated in figure 4-98a, during spring thaw. The sloughing occurs when the ice-filled soil thaws relatively rapidly. Impervious underlying frozen material prevents drainage of excess water in that direction. The resulting very wet, low shear strength soil therefore slumps or flows downward as illustrated in figure 4-98a. The effect can be intensified by earthquake accelerations. Emerging excess water may also move some soil downward by erosion. The phenomenon occurs typically in frost-susceptible fine-grained soils under conditions where sufficient moisture is available to build up substantial excess water in the annual frost zone in the form of ice lenses.

(2) Frost-sloughing is more common in cut slopes than fills because of the greater availability of moisture. However, it is not uncommon in embankments, especially if the slopes are made very steep. It is also more common and more severe on north-facing than south-facing slopes (in the Northern Hemisphere), probably because north-facing slopes tend to be wetter and although onset of thaw is delayed, its progress is fast once it starts. In wet cuts in which seepage emerges from the face, sloughing and erosion may also occur during non-frost periods.

(3) Slope flattening can control frost sloughing but may be very costly or not feasible. Drainage to reduce the amount of moisture available for ice segregation usually provides only partially effective control, especially when soils have strong horizontal stratification, and may be very expensive. Turf helps to control sloughing but its effects is marginal. Both the frost and non-frost types of slope problems can be controlled by blanketing the slope with granular pervious material as illustrated in figure 4-98c. The blanket material should be graded to act as a filter but also be sufficiently coarse-grained so that slow seepage can emerge from it without movement of particles. At the toe of slope the blanket should be carried a short distance below the adjacent surface as shown in figure 4-98c to avoid supporting the toe on material which will experience sufficient frost weakening in spring and to avoid loss of support under the toe by seepage erosion of fine-grained soil. The blanket functions in several different ways. It serves as a surcharge weight to reduce the amount of ice segregation per unit volume and hence the volume of water to be released from the frozen frost-susceptible material in spring. It serves as a semi-insulating layer, to reduce the amount of frost penetration into the frost susceptible material and to slow rate of thaw into this material in spring. Its loading effect serves to assist reconsolidation of frost-loosened material. It serves as a relatively high strength reinforcing material within the zone potentially involved in the sliding action. It also provides drainage. Because it is designed as a filter, seepage may emerge through it without loss of fine particles from the frost-susceptible zone or plugging of the voids in the blanket material.

(4) Successful use of both cinders and bank-run gravel as the blanketing material has been reported in numerous cases. However, gravel will be the normal blanketing material in arctic and subarctic areas. Good quality crusher run rock can also be used.

(5) Where frost and ordinary seepage sloughing and erosion of slopes are definitely anticipated in seasonal frost areas, protective blankets should be specified as part of the original design when they provide the most cost-effective approach. Although the blankets require initial extra expense, as compared with untreated slopes, maintenance costs resulting from unstable slopes can be eliminated. Blankets may also be used to correct unanticipated problems, but added expense to prepare the sloughed face of the slope will then be involved. Blanket thickness should be between 6 inches and 30 inches, with the larger thicknesses used for the most severe cases. In most cases 18 inches or 24 inches will be needed. Opportunity to use as little as 6 inches is expected to be rare in arctic and subarctic areas. Vertical to near-vertical slopes have been tried in highway cuts in Alaska in search of a more economical but still satisfactory solution. The wind-deposited silts in Alaska, when free of ground ice, have significant loess or loess-like properties and in areas of low precipitation have appreciable capacity for standing vertically for heights of 20 feet or more. However, a number of problems have been observed[52]. These include a tendency for spalling to occur in slabs about 4 in. thick, more on south-facing slopes than on north, attributable to such causes as moisture fluctuations and thermal stresses, collapse from undercutting by erosion or by loss of toe stability from moisture at the ditch level, and erosion or gullying from the top of slope downward, caused by discharge of surface run-off over the top during periods of heavy precipitation or snow melt. If the ditch must carry drainage flow, the slope is especially vulnerable and blockage of drainage by collapse materials may cause secondary damage. Slumping onto the roadway may be possible. Drifted snow and snow cast by snow removal equipment can cause a wet condition on thawing, not only in the ditch but to some extent on the face itself. Low precipitation with long periods of dry weather does not insure against occurrence of wet conditions at some period of the year. In-



U. S. Army Corps of Engineers

Figure 4-98. Slopes in frost and permafrost areas.

tercepting ditches above the top of cut can control discharge of moisture over the top, and may be needed regardless of degree of slope, but the consequences of seepage of water from the ditches into the soil directly behind the face may be more severe with the steep slopes. In fine-grained soil, especially silts, great care is required to avoid gullying and progressive soil erosion where the surface vegetative mat is cut, as by an intercepting ditch or at the cut slope, and surface flow of water occurs. Such erosion can involve large areas and must be corrected in its earliest stages. The savings in initial construction costs obtained by employing nearly vertical slopes as opposed to conventional flatter slopes must be balanced against such extra costs as providing wider ditch area to allow for spalling, erosion and sloughing, placing gravel or crushed rock at the toe when needed to insure stability at the ditch level, and maintenance efforts for periodic clean up and removal of slope-wasting materials, restoration of drainage, etc., which are not required when positive slope stability is provided at the start.

b. Sloughing and thaw settlement in areas of permafrost.

(1) When a cut is made in permafrost containing substantial amounts of ground ice, frost-sloughing and seepage erosion effects are intensified because of the potentially much larger volumes and deeper extent of deposits of ice and because of the irregular general settlement of the slope which may occur when this ice melts, as illustrated in figure 4-98b. If it is necessary to produce a reasonably stable slope during the initial construction, the amount of protective earth covering which will develop as permafrost degrades should be evaluated and a blanket thickness adopted which will not only control sloughing and erosion but ultimately limit further permafrost degradation. This will require the initial cut to be made with more or less conventional side slopes. The blanket should cover the full height of the cut slope.

(2) If the excess ice content of the natural soil is relatively low, the same protective blanket criteria as presented above for areas of seasonal frost should be used, except that blanket thicknesses should be in the range of 18 to 36 inches. As ice masses melt and drain away, the blanket may develop an irregular surface, but so long as the blanket remains intact, it will retain its function. Except in most northerly areas, it will seldom be economical to place sufficient thickness of blanket to contain thaw within the blanket. Some redressing of the slope may be done if needed, in future years.

(3) If the ice content of the permafrost is higher and the sloughing penetrates deep it may be necessary to use up to 3 to 5 feet of blanket material. This was done successfully to stabilize a sloughing slope in silt at the CRREL tunnel in permafrost at Fox, Alaska, for construction of the tunnel portal.

(4) If the excess ice content is very high it may be necessary to ultimately place a very substantial blanket designed to make up for the low amount of soil naturally present in the slope. Since relatively fine-grained moisture-holding soil is more effective than gravel for this purpose, the blanket may in this case consist of two layers an underlying zone of random fill and an overlying granular blanket not exceeding 3 to 5 feet in thickness. When possible, it is advantageous under these conditions to make the initial cut slope quite steep and to allow natural degradation and build-up of protective cover to occur for up to several years before placing the final protective blanket. Required combined thicknesses of blanket materials and natural cover for complete thermal stability may be computed as described in paragraph 4-2.

(5) Under conditions where a lengthy period of slope adjustment is acceptable, initial construction cost may be significantly reduced by going even further and making cuts with vertical or near-vertical slopes with wider than normal ditches, leaving the natural cover undisturbed and allowing the slope to seek a relatively stable condition by natural processes, at expense of greater maintenance costs. This method was tried in 1970 along the Trans-Alaska Pipeline Access Road from Livengood to the Yukon River, Alaska[195]. Cut faces contained up to 70 percent ice, as illustrated by figure 4-99. During summers following construction, melting of the ice caused the cut faces to assume quite irregular but gradually flatter slopes, accompanied by dropping of the organic mat and thawed soil down over the initially exposed ground ice to provide a progressively increasing thickness of protective cover as shown in figure 4-100. Considerable sloughing and down-slope erosion of fine-grained soils developed. Study of silt faces cut by gold mining operations in the Fairbanks area indicates that in areas of relatively warm permafrost, slope instability and adjustment under this approach may continue for many years or indefinitely, even after moderate-size trees have grown on the slope. Where significant ice is present, the slope may become very rough and unsightly. Periodic removal of silt from ditches and drainageways will be needed. If road-way ditches are allowed to fill, not only will drainage fail to function properly but snow and ice control in winter may become more difficult. Silt in run-off may also cause undesirable or unacceptable environmental effects unless it is prevented from entering streams which receive the drainage discharge. Where slopes are high the possibility that sloughing or slides may encroach on the pavement must be considered. Progressive gullying and erosion where the cut slope intercepts surface drainageways must be corrected expeditiously. Ultimately, the natural stabilization processes may have to be supplemented.

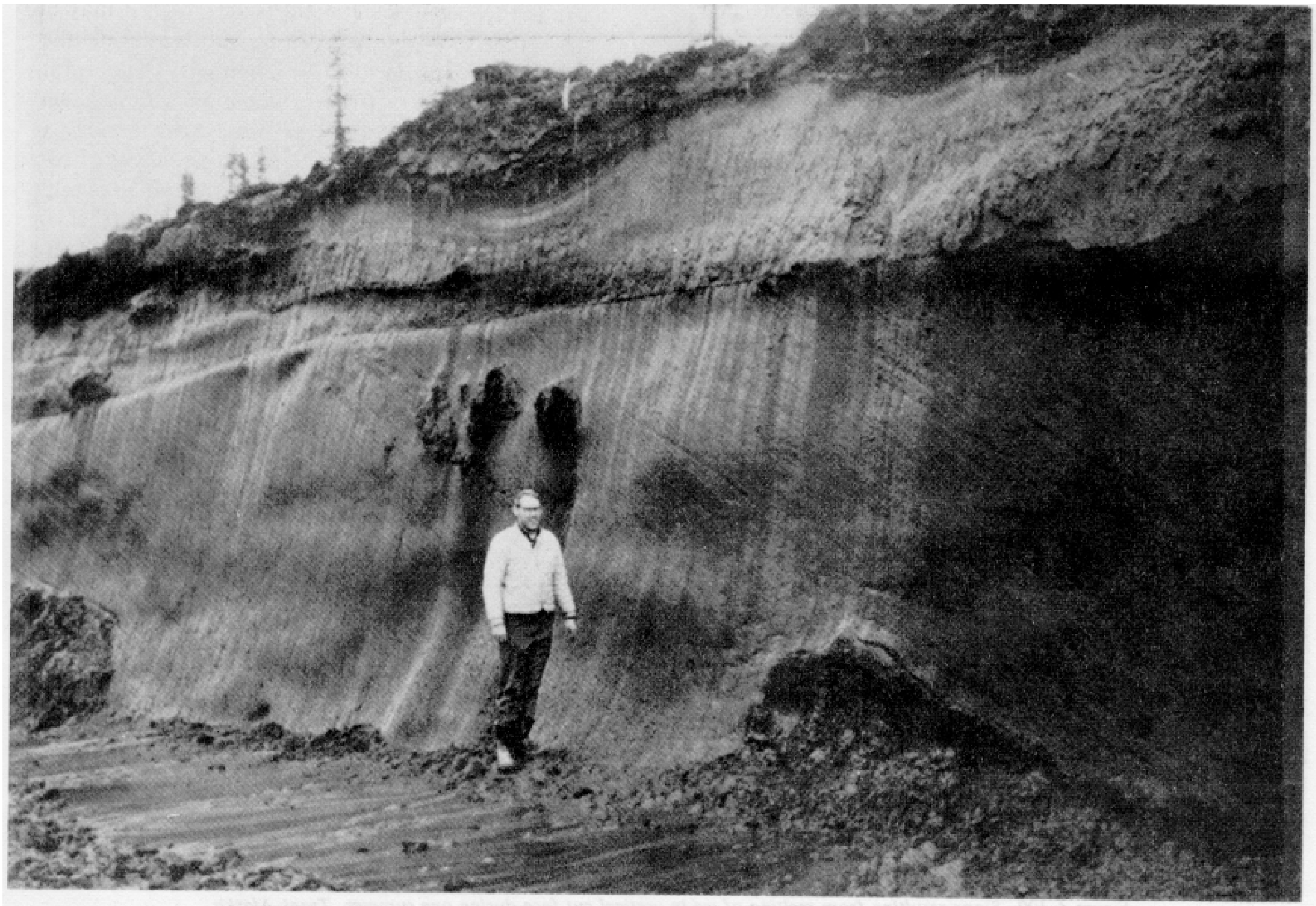


Figure 4-99. Ice exposed in vertical cut for Trans-Alaska Pipeline Access Road between Livengood and the Yukon River, Alaska, April 1970.

Figure 4-100. Slope resulting from melting of ice in vertical cut face during one summer, Trans-Alaska Pipeline Access Road between Livengood and the Yukon River, Alaska, August 1970.